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SUSPENSION BRIDGES AND CANTILEVERS

THEIR ECONOMIC PROPORTIONS AND LIMITING SPANS

Submitted in partial fulfilment of the requirements for the Degree of Doctor of Philosophy, in the Faculty of Pure Science, Columbia University.

BY

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PREFACE.

In recent engineering literature there is frequent reference to the question of the relative adaptability of the cantilever and suspension bridges to long span construction and to the dearth of adequate data from which the limiting and economic spans for the two bridge-types might be deduced. In order to supply this deficiency and to determine as definitely as practicable the length of span at which the suspension bridge becomes economically superior to the cantilever, the author has undertaken the investigations which are summarized in the following pages.

In connection with these investigations there have arisen several subsidiary problems of design. It was found necessary to determine the economic rise-ratio for suspension bridges, the minimum depth of stiffening trusses for

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adequate rigidity, the economic depth of stiffening truss, the best span-ratios and the minimum width for cantilevers and allied questions of design or construction. The solutions of these problems, together with outlines of the methods of designing the different parts of the bridge structures are included in this book.

The author desires to express his indebtedness to Professor W. H. Burr of Columbia University for his valuable suggestions and helpful guidance, to Professor C. N. Little of the University of Idaho for help in correcting the manuscript and other kind assistance, and to the Department of Bridges of New York City for considerable information and data freely placed at the author's disposal.

COLD SPRING ON THE HUDSON,
August 1, 1911.

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SUSPENSION BRIDGES AND CANTILEVERS

CHAPTER I

INTRODUCTION

ART. 1

STATEMENT OF PROBLEM AND PROPOSED METHOD OF INVESTIGATION

EACH type of bridge construction has some limiting span-length which it cannot, physically, exceed. This *maximum span* may be defined as the length at which the ratio of the intrinsic weight to the applied weight becomes infinite. In other words, it is the length at which the structure cannot carry any load in excess of its own

weight; any attempt to increase the load resulting in members of infinite cross-section. Under the stress of necessity, the span of any type of bridge may be pushed as close as may be desired to the maximum span-length; but this can be done only at a very great sacrifice of economy, as the cost of the structure increases very rapidly when the span approaches the limiting value. The length of the maximum span for any form of bridge may be determined with sufficient definiteness from theoretical considerations applied to the data of actual designs; but it must be borne in mind that the results of such determination are subject to expansive revision when the methods of design or the materials of construction undergo improvement.

In addition to a maximum span-length, each bridge type has an *economic range of spans*, within which it will be less costly than any other form of construction. If, for any two comparable types of structure, a *span of*

equal cost be determined, that span will be the inferior economic limit for one of the bridge-types and the superior economic limit for the other.

The term *economic span* will be used to designate the span at which the cost of a bridge would exceed the capitalized value of its usefulness to the builders. Although this is a problem of great practical significance, it does not lend itself to accurate treatment in any general manner owing to the large possible variations in local conditions, such as the magnitude and financial importance of the traffic expected, the cost of real estate for land spans and approaches, and the difficulty encountered in locating suitable foundations. The results of a general determination of the economic span, as here defined, will therefore be of doubtful practical value except for purposes of illustration or comparison.

The maximum and economic span-lengths for bridges of ordinary span have been fixed pretty definitely by

the results of numerous designs and comparative estimates. We may take our values of these limiting lengths from the consensus of opinion among engineers as evidenced by their uniformity of practice. Thus the ordinary truss bridge finds its range of usefulness between the limiting spans of about 120 and 550 ft., there being but three truss spans exceeding the latter value. Below this range, the steel girder or concrete arch is more economical; above these limits the steel arch enters into competition. The latter construction, in turn, ceases to be economical at about 800 ft., although the proposed Hell Gate Arch is to have a span of 978 ft.

For longer spans, the selection of a bridge structure narrows down to a choice between the *suspension bridge* and the *cantilever*. The relative economy of these two forms of construction has long been a mooted question. It is true that the longest span in existence, viz., the Forth Bridge, is of the canti-

lever type, but it is a question whether the selection of this type was not a mistake. "It is not a design which would ever be imitated. Its proportions are very injudiciously taken, and there is a failure to reach the degree of economy which ought to exist even in the cantilever."[1]

In the preliminary investigations for the Quebec Bridge, the Phoenix Bridge Company made comparative estimates of a cantilever and a suspension bridge for the 1800-ft. span. "Although the cantilever type exhibited the more economic results of the two as the members were then computed, at the present time the economy of the adopted type is not so clear as it was originally thought to be."[2] The collapse of that ill-fated structure, and the investigations following the completion of the Queensboro Bridge have shaken

[1] W. H. Burr, Proceedings of the Engineers' Club of Philadelphia, December, 1899.

[2] Editorial in *Engineering Record*, Sept. 5, 1908.

the general confidence in the cantilever type of construction and have directed the attention of engineers to the more adequate design of compression members in such structures. When these members are designed in accordance with the recent disclosures, it is a question whether the previously accepted economic limit for cantilevers, viz., about 2000 ft., will not have to be considerably reduced.

The limiting *economic span* for cantilevers is placed by Prof. Burr [1] at 2000 ft. for railway bridges and at 1400–1600 ft. for highway bridges; by Prof. Merriman [2] at 1500 ft.; and by Prof. Melan [3] at 500 meters. Gustav Lindenthal [4] (M. Am. Soc. C.E.) would

[1] Proceedings of Engineers' Club of Philadelphia, December, 1899. *Ancient and Modern Engineering* (New York, 1903), p. 177.

[2] *Roofs and Bridges* (New York, 1905), Part IV, pp. 110, 155.

[3] *Handbuch der Ingenieur-Wissenschaften* (Leipzig, 1906). II. Band, V. Abteilung, s. 206.

[4] *Engineering* (London), December 19, 1890 also Proceedings Am. Soc. C. E., Sept. 21, 1904.

use the cantilever for spans between 500 and 2000 ft., the steel arch between 2000 and 4000 ft., and the suspension bridge for all greater spans. Jos. Mayer [1] (M. Am. Soc. C.E.) considers the minimum economic span for the suspension type to be 800 ft. for highway bridges and 1300 ft. for railway bridges. No mention is made of data upon which these estimates are based.

The question of *maximum span* has also been under discussion. In 1894, a prominent bridge engineer declared before a congressional committee that "a bridge of 2800 ft, would be just capable of holding its own weight without carrying any live or moving load." [2] A few months later, a board of engineers approved a design for a suspension bridge of 3100-ft. span and showed its "feasibility of manufacture and cost." This Board, appointed by act of Congress to determine the practicability of a long-span bridge across

[1] Proceedings Am. Soc. C. E., October, 1904.
[2] *Engineering Record*, Nov. 18, 1899.

the Hudson River at New York City, and consisting of Major C. W. Raymond, W. H. Burr, G. Bouscaren, Theodore Cooper and Geo. S. Morison, conducted a series of investigations and reported that, at the proposed site, a 2000-ft. clear-span cantilever could be built for $27,000,000, a 3100-ft. clear-span cantilever for $51,000,000, and a 3100-ft. clear-span suspension bridge for $31,000,000.[1] These results indicate that the maximum practicable limit for suspension bridges is above 3000 ft., and that the economic limit for cantilever bridges is far below that value.

In the same year the Secretary of War directed the formation of "a Board of Officers of the Engineer Corps who shall investigate and report their conclusions as to the maximum length of span practicable for suspension bridges and consistent with an amount

[1] Senate Executive Documents, 53d Congress, 3d Session, No. 12, Report of Board of Engineers on the N. Y. and N. J. Bridge, Aug. 23d 1894.

of traffic probably sufficient to warrant the expense of construction." Assuming that the bridge of maximum span is supported by sixteen 21½-inch cables, and has to carry a uniform live-load of 27,540,000 lbs.÷L. the Board obtained a value of $L=4335$ ft. for the practical maximum span.[1] The above assumptions, however, were rather arbitrary; any other combination of assumed values for cable-section and loading would have resulted in a different value for the maximum span-length.

Although comparative designs of the two types of long-span bridges have been prepared in individual instances for the particular local conditions obtaining, the writer has been unable to find any general comparison of the two bridge-types for the purpose of establishing their relative economic or

[1] Senate Executive Documents, 53d Congress, 3d Session, No. 12, Report of Board of Engineer Officers to Make Investigations of Certain Bridges, Sept. 29, 1894.

limiting spans. He has therefore undertaken the determination of these values, namely:

1. The maximum practicable span for the suspension-bridge form of construction.

2. The maximum practicable span for cantilevers.

3. The maximum economic span for suspension bridges.

4. The maximum economic span for cantilevers.

5. The span of equal cost for the two types; in other words, the span at which the cantilever ceases to be economically superior to the suspension bridge.

The method proposed for the solution of these problems is to prepare designs and estimates of a wide range of cantilevers and suspension bridges and to deduce therefrom the laws of variation of weight and cost with length of span. The relations between span and weight, thus established, will fix the maximum feasible span for

each form of construction. The cost-curves of the two types will indicate their comparative economy at different spans as well as the critical span of equal cost. Finally, an estimate of the maximum probable traffic returns, compared with the costs of different spans, will determine the economic limiting length for each type.

The cost of a structure for a given span will, of course, be affected by local conditions such as prices of material, specified unit stresses, depth of foundations, etc. In order that the comparison between the cantilever and the suspension bridge may be an absolutely fair one, it is essential that all such arbitrary or varying factors be chosen with extreme care and be kept exactly the same for both types of construction. Furthermore, in order that the results for the limiting spans may be the true maximum values, it is necessary to carefully determine and use the most favorable proportions, material and form of construction for each design.

TABLE I.—NOTABLE

Date.	Name.	Location.	Engineer.
1903	Williamsburg..	East R., N. Y....	L. L. Buck..
1883	Brooklyn.....	East R., N. Y....	Roebling....
1909	Manhattan....	East R., N. Y....	Dept. of Br..
1869	[1] Niagara	Niagara Falls....	Keefer.....
1867	Ohio R.....	Cincinnati, O....	Roebling....
1851	[2] Niagara	Lewiston, N. Y ..	Serrell.....
1900	Miampimi.....	Mexico.....	.....
1848	[3] Wheeling.....	Ohio R.. W. Va..	Ellet.....
1903	Elizabeth.....	Budapest.....	Dept. of Br..
1834	[4] Freiburg.....	Switzerland.....	Chaley.....
1855	Niagara Ry...	Niagara Falls....	Roebling....
1899	Niagara	Lewiston, N. Y..	R. S. Buck..
1900	Rochester.....	Ohio R., Pa.....	.....
1877	[5] Point.....	Pittsburg.....	Hemberle...
1902	Vernaison.....	France.....	.....
1896	E. Liverpool ..	Ohio.....	.....
1864	Clifton	Bristol, Eng.....	.....
1845	Lancz.....	Budapest.....	Clark.....
1855	Morgantown ..	W. Virginia.....	.....
1825	Menai	Wales.....	Telford .....
1905	Villefranche...	France.....	(Gisclard)...
1904	Caperton	W. Virginia.....	Cooney .....
1868	[6] Moldau.....	Prague.....	.....
1852	[7] Charleston....	W. Virginia.....	Ellet.....
1818	Tweed.....	Berwick, Eng....	Brown.....
.....	Grand Ave....	St. Louis.....	.....
1862	Lambeth	London.....	.....
1826	[8] Conway	Chester, Eng.....	.....

[1] Strengthened 1888. Wrecked and rebuilt 1889. Replaced by arch 1896.
[2] Removed 1864.
[3] Rebuilt 1854 and 1862.
[4] Two cables added 1881.
[5] Reconstructed 1905.
[6] Rebuilt with cables 1900.
[7] Failed 1904.
[8] Rebuilt 1904.

Suspension Bridges.

Span.		Rise $=f$	Cables.		Truss.		Loading.		
l	l_1		No.	Size.	D'p'h	W't'	L.L.	D.L.	D.L./L.L.
1600	596	177	4	18¾″	40	118	6300	16620	2.6
1595	*930	†128	4	15⅝	9, 17	85	2600	8200	3.2
1470	*725	160	4	21¼	24	122	8000	18000	2.2
1268									
1057	*281	†90	4	12½	28	52	3600		
1042	....	87	8	.....	.....	21			
1030	....	†	2	.....	5	....	100	300	3.0
1010	....	....	4	8					
951	145	95	4	20″E.B.	23	66	5500	14500	2.6
870	174	63	6	2, 8	5	21			
821	....	†54	4	10¼	16	25	1800	1800	1.0§
800	34	53	4	10	14	28	2400	2850	1.3
800	*416	72	2	7	18				
800	145	88	2	8″E.B.‡	8	34	1700		
764	*172	†	2	.....	4	17	680	600	0.9
705	....	....	2	7⅛					
702	....	....	..	Chains					
663	*285	48	4	Chains					
608	150	42	6	3¼	5	20	2000		
578	*260	43	16	Chains	.....	.....	.....	2500	
512	128	†	2	‡Wire	0	20	2000	2000	1 0
510	*	37	2	1½	0	6	120	100	
482	.....	.....	8	Chains	.....				
478	....	25	4	3¼	0	17			
450	....	30	12	Chains					
400	*150	40	4	‡Chains	.....	60			
363	.....	.....	4.	‡Chains					
327	....	22	...	Chains					

* Provided with suspenders in side spans.
† Stiffened with diagonal stays.
‡ Braced cable construction.
§ Railway bridge. Replaced by arch in 1897.

CHAPTER II

STUDY OF SUSPENSION BRIDGES

ART. 2

SUSPENSION BRIDGES

THE economic utilization of the materials of construction demands that, as far as possible, the predominating stresses in any structure should be those for which the material is best adapted. The superior economy of steel in tension and the uncertainties involved in the design of large-sized compression members point emphatically to the conclusion that the material of long-span bridges, for economic designs, must be found to the greatest possible extent in tensile stress. This requirement is best fulfilled by the suspension-bridge type.

The superior economy of the sus-

pension type for long-span bridges is due fundamentally to the following causes:

1. The very direct stress-paths from the points of loading to the points of support.

2. The predominance of tensile stress.

3. The highly increased ultimate resistance of steel in the form of cable-wire.

With the exception of the Forth Bridge, the Queensboro Bridge and the Quebec Bridge (under construction), all structures exceeding 1000 ft. in span have been suspension bridges. Table I gives a list of the most notable structures of this type, with their principal dimensions. It is seen from this table that all suspension bridges erected after what might be called the experimental period (1796–1876) have a minimum span of about 800 ft. and a maximum span of 1600 ft.

In order to secure information needed in determining the maximum and economic spans for suspension bridges,

in addition to the data supplied by existing structures, the writer has undertaken the design of three suspension bridges, having span-lengths of 1500, 2250 and 3000 ft., respectively. To justify the large expenditure involved in structures of this magnitude, they will be assumed to be railroad bridges, with additional provision for electric cars, driveway and footwalks.

Before we can proceed with the designing of the structures, it is necessary to find the most favorable solutions of the following problems:

1. The choice between wire-cable and eye-bar construction.

2. The economic ratio of cable-rise to length of span.

3. The best ratio of depth of stiffening truss to length of span.

Art. 3

Wire Cable *vs.* Eye-bars

One of the first questions to be decided in the design of a suspension

bridge is the choice between a steel-wire cable and a chain of eye-bars for the principal carrying member. The latter enables the bracing for the prevention of deformation under moving load to be incorporated in the cable system; the other requires a separate stiffening truss for the reduction of these deflections. A third method of bracing the suspension bridge against deformation is the introduction of diagonal stays between the towers and the roadway, as was done in the Ohio River, Brooklyn and Vernaison Bridges. This method, however, has been abandoned in recent designs as the stays have been shown to be of doubtful utility and, furthermore, fail to act in unison with the cable and suspenders under changes of temperature.

The earliest suspension bridges were built with chains. James Finley, the pioneer builder of suspension bridges in America, used common wrought-iron chains for all of his bridges from the 70-ft. span at Uniontown, Pa.

(1796), up to his greatest achievement, the 309-ft. span of the Schuylkill Bridge (1808). In 1818, Brown substituted a chain of eye-bars, bolted together, in building the Tweed Bridge of 450-ft. span. This construction was followed in the Menai Bridge, built by Telford in 1825, and the Hammersmith Bridge (London) built by Clark in 1827. The following year an advance was marked by the use of open-hearth steel for the chains of the Karl Bridge at Vienna (312 ft. span). Steel eye-bar chains remained in use for bridges of constantly increasing span, including Brunel's Hungerford Bridge (London, 1845), Clark's Bridge at Budapest of 663 ft. span (1845), and the Clifton Bridge of 702 ft. span (Bristol, 1864).

In the meantime, John A. Roebling was at work at Saxonburg, Pa., inventing and developing the manufacture of wire rope. He soon conceived the possibility of its application in the construction of suspension bridges, in

place of the eye-bar chains, on account of its superior strength and ease of erection. Beginning with the use of wire cable for the suspension of canal aqueducts, he proceeded to apply it to more ambitious structures. In 1848, Chas. Ellet built the wire-cable bridge over the Ohio River at Wheeling with a span of 1010 ft. which was blown down in 1854. In 1851, Roebling commenced his 821-ft. suspension bridge over Niagara Falls, the first and only suspension structure to be built for heavy railroad traffic. In 1867, he completed the Covington and Cincinnati Bridge over the Ohio River, with a span of 1057 ft. These achievements of Roebling, however, were but a preliminary training for "the monumental work that was to cost him his life while crowning it with glory." When he presented his plans for spinning his steel wires over the vast span of the Brooklyn Bridge, he had to defend his ideas against the scoffing of the whole world and had to fight his

opponents inch by inch before the right to try was given him. The bridge was finished by his son in 1883, and held the record for length of span (1595.5 ft.) until the Williamsburg Bridge (1600 ft.) was completed by L. L. Buck in 1903. With the third span across the East River, the Manhattan Bridge of 1470 ft. span, opened in 1909, we are brought to the present day in the history of suspension bridges.

Since the time of Roebling, wire cables have been used in all suspension bridges with but one exception: the Elizabeth Bridge over the Danube River at Budapest (1903). Its span of 951 ft. is the largest of any suspension bridge outside of America.

In nearly all of the above bridges, a stiffening truss is the means employed to prevent the deformations due to live-load, wind and changes of temperature. Another practicable method is to build the cable as a trussed structure like an inverted two-hinged or three-hinged arch. This method was

used in the Point St. Bridge at Pittsburg (span = 800 ft.) which was built by Hemberle in 1877. In this bridge the cable is composed of 8-inch eye-bars and is trussed on its upper side by bracing connecting it to two straight chord-members running from the ends to the middle of the cable.

In 1894, Gustav Lindenthal offered a design for a suspension bridge over the Hudson River, with a clear-span of 3100 ft., in which he used two pairs of parallel cables connected by a system of bracing in a vertical plane.[1] He proposed building the cables of pin-connected wire links, claiming for such construction the advantages of accurate work and close inspection in the shop, rapid erection, and possibility of varying the cable-section as required.

In preparing the design of the Manhattan Bridge over the East River, the New York Department of Bridges followed Lindenthal's scheme, using

[1] Report of Board of Engineer Officers on the N. Y. and N. J. Bridge, 1894. Appendix D.

braced cables built up of eye-bars. The following year, with a change of city administration, that plan was abandoned and the structure redesigned with wire cables. In September, 1904, Lindenthal explained and defended his design in a paper read before the American Society of Civil Engineers,[1] which gave rise to a long series of very fruitful discussions.[2] The principal advantages claimed for the two types may be summarized as follows:

Advantages of the Braced-cable Type of Suspension Bridge

1. By having the greatest depth of the bracing at the one-quarter points, where the maximum moments occur, the stiffness of the bridge with a given expenditure of material is greatly increased.

2. The small depth along the middle third of the span reduces the temperature stresses.

[1] Transactions Am. Soc. C. E., Sept. 21, 1904.
[2] Ibid., Oct., 1904 to Mar., 1905.

3. The stiffened-cable construction saves one chord of the truss as the cable itself forms the upper chord.

4. The section of an eye-bar chain may be varied with the stress, whereas the entire wire cable must have the maximum section.

5. Greater weight, if it does not increase the cost, is an advantage in a bridge, as it serves to increase the rigidity of the structure.

6. The pin-connections facilitate the speedy erection of the cable.

Disadvantages of the Braced-cable Type

1. An unpleasant appearance is produced by the lattice-work up in the air.

2. Since the bottom chords of the bracing run to the top of the towers, special wind-chords at the floor-level become necessary for lateral stiffness, and these may be as heavy as the bottom chords themselves.

3. It is impossible to calculate the

stresses accurately on account of the difficulty in adjusting the members. The safe working stresses should therefore be reduced by at least 5 per cent.

4. The braced-cable type exposes a large area to the wind at the highest elevation, thereby greatly increasing the wind-stresses in the bracing and in the towers.

5. If the floor is not stiffened vertically, every suspender receives a heavier concentrated load and every shock from moving loads is transmitted directly to the cables instead of being absorbed by the floor system.

6. The braced system introduces many elements of uncertainty and complexity in the structure, and the history of bridge design shows that "the lines of progress have been in the direction of eliminating uncertain elements and holding fast to those features which secure certainty in the determination of stresses."

7. The practical difficulty, hazard and expense of making satisfactory

connections between a cable and the web-members of an overhead bracing system preclude the use of wire cables, and to abandon the wire cables is to abandon all the essential advantages of the suspension bridge. The superior economy of the wire cable over eye-bar construction rests on the following considerations:

(*a*) Steel wire with an elastic limit of 180,000 lbs. per sq.in., is obtainable at a cost of but twice as much per pound as nickel-steel eye-bars with one-fourth the elastic limit.[1]

(*b*) The eye-bar heads and pins add about 25 per cent to the weight of the cable.

(*c*) An eye-bar chain will therefore weigh about four times as much, and cost about twice as much as a wire cable to carry an equal load, if the same factor of safety is to be maintained.

(*d*) The increased dead-weight of

[1] Cf. R. S. Buck and Jos. Mayer, Trans. Am. Soc. C. E., Oct., 1904.

the cable, when eye-bars are used, results in increased stresses in cable, towers and anchorages.

(*e*) The wire cable is self-supporting during erection and all the problems involved have been worked out and successfully demonstrated. The eye-bars, on the other hand, would require a temporary supporting cable; and the manufacture and erection of eye-bars of suitable size for very long spans present many unsolved difficulties.

Many of the advantages enumerated above, particularly those bearing on the economy of the respective types, practically balance each other, so that, in consequence, there is really no material difference between the costs of the two forms of construction. In a comparative design of the two types for a proposed bridge at Cologne, of 722 ft. span, O. Erlinghagen found that the eye-bar chain would require about twice as much material as the wire cable but, on account of the difference in unit prices, the total cost of the two

structures was almost exactly the same.[1] Jos. Mayer (M. Am. Soc. C.E.) compared the two types for a span of 3000 ft., and found the braced-cable type to weigh 5000 lbs. more per linear foot than the other.[2] Prof. J. Melan found the difference of cost between chain and cable for the Elizabeth Bridge at Budapest to be negligible, the eye-bar chain being adopted for other considerations.[3] The two designs for the Manhattan Bridge showed a difference in cost of 7 per cent in favor of the eye-bar type, although the chain in the latter weighed 2½ times as much as the wire cable.[4]

Omitting from consideration the possibility of using the overhead system of bracing in conjunction with a wire cable, a construction which would combine the essential advantages of

[1] *R. R. Gazette*, Nov. 20, 1903.

[2] Transactions Am. Soc. C. E., Vol. 48, p. 371.

[3] Transactions Am. Soc. C. E., Feb., 1905.

[4] *Engineer* (London), Aug. 28, 1903.

both systems but of which the feasibility has not yet been demonstrated, and without presuming to decide between the relative advantages of the two types of cable construction, the writer will adopt the *wire-cable type* in the present investigations for the following reasons:

1. As both types are practically equal in cost, this choice will not affect the results for the economic lengths of span.

2. As the wire cable affords a great saving in weight, it will yield a larger value for the maximum practicable span.

3. As this choice conforms with the accepted past and present practice in suspension bridge design, and as no radical changes need be expected in the proximate future, the results obtained will be better adapted for comparison with the data of existing structures and of greater value in estimating future designs.

Art. 4

The Economic Ratio of Rise to Span

(*a*) *Cost of Cable.* Let n = the ratio of the rise (f) to the span (l) of the cable. The total load per linear foot carried by the cable consists of two parts: g_0 = the weight of the cable itself, and g_1 = the weight of the total suspended (dead and live) load. As is well known, if α is the inclination of the cable at the towers, the maximum tension in the cable will be

$$T = H \cdot \sec \alpha = (g_0 + g_1)\frac{l^2}{8f}\sqrt{1+16n^2}. \quad (1)$$

The weight of the cable per *horizontal* linear foot is given by

$$g_0 = (1+\tfrac{8}{3}n^2)\frac{T}{s_0}\cdot\gamma_0, \quad . \quad . \quad (2)$$

where s_0 is the unit working stress and γ_0 is the weight per linear foot per square inch of cross-section. Sub-

stituting the value of T from Eq. (1), Eq. (2) may be written,

$$g_0 = k(g_0 + g_1) \cdot l \cdot \frac{1}{n}\sqrt{1+16n^2} \quad . \quad (3)$$

where

$$k = (1 + \tfrac{8}{3}n^2)\frac{r_0}{8s_0} \quad . \quad . \quad . \quad (4)$$

The factor $(1+\frac{8}{3}n^2)$, for all practical values of n, varies only from the value 1.02 to 1.05; hence the quantity k may be considered as practically independent of the rise-ratio n. The solution of Eq. (3) gives

$$g_0 = g_1 \frac{kl\frac{1}{n} \cdot \sqrt{1+16n^2}}{1 - kl\frac{1}{n} \cdot \sqrt{1+16n^2}} \quad . \quad (5)$$

If L_0 = the total horizontal length of cable and c_0 = the cost per pound of the cable material, the total cost of the cable will be,

$$C_0 = L_0 \cdot c_0 \cdot g_0.$$

Substituting the value of g_0 from Eq. (5), we obtain

$$C_0 = a \cdot (g_1 l^2) \cdot \frac{k \frac{1}{n} \sqrt{1+n^2}}{1 - kl \frac{1}{n} \cdot \sqrt{1+n^2}} \quad . \quad (6)$$

where

$$a = \frac{L_0}{l} \cdot c_0. \quad . \quad . \quad . \quad (7)$$

Numerical Values. Allowing for the weight of cable wrapping, a mean value for γ_0 is 3.5. Hence, by Eq. (4), for a range of

$$\frac{f}{l} = \frac{1}{7} \text{ to } \frac{1}{10},$$

we may take,

$$k = \frac{0.455}{s_0}. \quad . \quad . \quad . \quad (4')$$

If $c_0 = 15$ cents per lb. and $\frac{L_0}{l}$ has its minimum value of 1.5 (for no suspenders in side spans),

$$a = 22.5. \quad . \quad . \quad . \quad . \quad (7')$$

If the side-spans are suspended, $\frac{L_0}{l}=2$, hence

$$a=30.0 \quad . \quad . \quad . \quad . \quad (7'')$$

(*b*) *Cost of Suspension Rods.* If L_2 is the total length of bridge provided with suspension rods, the total load carried by them will be $g_1 L_2$. Since the cable is a parabola, the average length of the rods is $f/3$. If s_2 is the unit stress, γ_2 the unit weight, and c_2 the unit cost, the total cost of the rods will be

$$C_2=c_2 \cdot \frac{g_1 \cdot L_2}{s_2} \cdot \gamma_2 \cdot \frac{f}{3}$$

This may be written

$$C_2=b \cdot (g_1 l^2) \cdot n \quad . \quad . \quad . \quad (8)$$

where

$$b=\frac{c_2 \gamma_2}{3 s_2} \cdot \frac{L_2}{l}. \quad . \quad . \quad . \quad (9)$$

Numerical Values. If $s_2=30{,}000$ lbs. per sq.in., $\gamma_2=3.4$, $c_2=12$ cents per

lb., and $L_2/l=1$ (i.e. no suspenders in the side-spans), then

$$b=.00045. \quad . \quad . \quad . \quad (9')$$

If $\frac{L_2}{l}=2$ (side-spans suspended), then

$$b=.00090. \quad . \quad . \quad . \quad (9'')$$

(*c*) *Cost of Towers.* With the entire bridge fully loaded, the total compression in each tower will be twice the end-shear in the cable or

$$P_3=(g_0+g_1)\cdot l. \quad . \quad . \quad (10)$$

If s_3 is the working intensity of stress, the required section for each tower is

$$A_3=\frac{P_3}{s_3}\cdot=\frac{(g_0+g_1)l}{s_3}, \quad . \quad (11)$$

or, substituting the value of g_0 from Eq. (5),

$$A_3=\frac{g_1 l}{s_3}\cdot\frac{1}{1-kl\cdot\frac{1}{n}\sqrt{1+16n^2}}. \quad (12)$$

Let γ_3 = the weight per linear foot of tower per square inch of cross-section, c_3 = the unit cost of the material, $\tau\cdot f$ =

total height of tower; then the total cost of the two towers will be

$$C_3 = 2 \cdot A_3 \cdot \gamma_3 \cdot \tau \cdot f \cdot c_3,$$

or, substituting the value of A_3 from Eq. (12),

$$C_3 = c(g_1 l^2) \cdot \frac{n}{1 - kl\frac{1}{n} \cdot \sqrt{1+16n^2}}, \quad (13)$$

where

$$c = 2\tau \cdot \frac{\gamma_3 \cdot c_3}{s_3} \quad . \quad . \quad . \quad . \quad (14)$$

Numerical Values. A comparative study of existing structures yields the following mean values for the constants: $\gamma_3 = 6.5$ (including diagonal members, details, etc.), $c_3 = 5.6$ cents per lb. (price of structural steel and erection), $s_3 = 8000$ lbs. per sq.in. (for the direct stress exclusive of bending stresses), $\tau = 1.6$ (a mean value for the longer spans). With these values, Eq. (14) gives

$$c = .0145. \quad . \quad . \quad . \quad (14')$$

Thus far, in the analysis, there is nothing essentially new. We now proceed to apply the above results to our immediate purpose.

Total Cost. The total cost of the structure, exclusive of truss, anchorages, substructure, etc. (which are independent of n), may be obtained by adding together the right-hand members of Eqs. (6), (8), (13), giving the result,

$$C=g_1l^2\left[a\cdot\frac{k\cdot\frac{1}{n}\cdot\sqrt{1+16n^2}}{1-kl\cdot\frac{1}{n}\cdot\sqrt{1+16n^2}}\right.$$
$$\left.+bn+c\cdot\frac{n}{1-kl\cdot\frac{1}{n}\cdot\sqrt{1+16n^2}}\right],$$

or, expanding the radical,

$$C=g_1l^2\cdot\left[\frac{ak(1+8n^2-32n^4+\ldots)+cn^2}{n-kl(1+8n^2-32n^4+\ldots)}+bn\right]. \quad (15)$$

The condition for a minimum total cost then becomes

$$\frac{\partial C}{\partial n}=g_1l^2\cdot$$
$$\cdot\left[\frac{\begin{matrix}(n-kl-8kln^2+32kln^4-\ldots)\\ .(2cn+16akn-128akn^3+\ldots)\\ -(cn^2+ak+8akn^2-32akn^4+\ldots)\\ .(1-16kln+128kln^3-\ldots)\end{matrix}}{n^2-2kl(n+8n^3-32n^5+\ldots)+k^2l^2(1+16n^2)}+b\right]=0.$$

Clearing of fractions, and arranging the terms according to the ascending powers of n, we obtain,

$$(bk^2l^2-ak)-(2ckl+2bkl)n+(8ak+b+c+16bk^2l^2)n^2 -(16bkl)n^3-(96ak)n^4+\ldots=0. \quad (16)$$

It may be readily shown that the terms containing n^3 or higher powers of n are absolutely negligible in comparison with the remaining terms, and they will therefore be dropped from the equation. Solving the resulting quadratic equation for n, we obtain

$$n=\frac{(b+c)2kl\pm\sqrt{4(b+c)ak+32a^2k^2 +k^2\cdot(32abk+4bc+4c^2)l^2-64b^2k^4l^4}}{2(b+c)+16ak+32bk^2l^2} \quad (17)$$

On account of the relatively small values of b and k, the terms containing bk^2 and b^2k^4 may be neglected without any appreciable error for any feasible length of span. The equation is thus simplified to

$$n=\frac{2k(b+c)\cdot l\pm\sqrt{4(b+c)ak+32a^2k^2+4ck^2(b+c)l^2}}{2(b+c)+16ak} \quad (18)$$

This equation shows:

1. That the economic rise-ratio (n) increases with the length of span.

2. That the economic rise-ratio increases with any increase in the cable-factors (a, k) and decreases with any increase in the tower-factor (c) or the suspension-rod factor (b). See Eqs. (4), (7), (9), (14).

Substitution of Numeral Values. Case I. No suspenders in the side-spans. By Eqs. (4′) (7′), (9′) and (14′)

$$k=\frac{.455}{s_0} \qquad b=.00045.$$

$$a=22.5 \qquad c=.0145.$$

Substituting these values in Eq. (18), we obtain

$$n=\frac{.0299l\pm\sqrt{2.961s_0+16210+.000867l^2}}{.0658s_0+360} \tag{18a}$$

If $s_0=60{,}000$ lbs. per sq.in., this becomes

$$n=.0070l+\sqrt{.0105+.000047l^2}, \tag{18′a}$$

where l is the span-length in thousands of feet. Hence, if

$$\left.\begin{array}{ll} l=0, & n=.103=\dfrac{1}{9.7} \\ l=1000', & n=.110=\dfrac{1}{9.1} \\ 2000', & n=.118=\dfrac{1}{8.5} \\ 3000', & n=.126=\dfrac{1}{7.9} \\ 4000', & n=.135=\dfrac{1}{7.4} \end{array}\right\} \text{Economic ratios of rise to span.}$$

Case II. Suspended side-spans.

By Eqs. (4′), (7″), (9″) and (14′),

$$k=\frac{0.455}{s_0} \qquad b=.0009,$$
$$a=30 \qquad c=.0145.$$

Substituting these values in Eq. (18), we obtain

$$n=\frac{.0308l\pm\sqrt{4.06s_0+28{,}800+.000894l^2}}{480+.0677s_0} \tag{18b}$$

If $s_0 = 60{,}000$ lbs. per sq.in., this becomes

$$n = .0068l \pm \sqrt{.0133 + .000044l^2}, \quad (18'b)$$

where l is the span-length in thousands of feet. Hence, if

$$\left.\begin{array}{ll} l = 0, & n = .116 = \frac{1}{8.7} \\ l = 1000', & n = .123 = \frac{1}{8.2} \\ 2000', & n = .130 = \frac{1}{7.7} \\ 3000', & n = .138 = \frac{1}{7.3} \\ 4000', & n = .146 = \frac{1}{6.9} \end{array}\right\} \text{Economic ratios of rise to span.}$$

The above results justify the value of $n = \frac{1}{8}$ generally recommended for spans of the usual length.

Working Stress in the Cable

In the preceding treatment, a value of 60,000 lbs. per sq.in., was assumed for the unit stress in the cable. This conforms to generally approved practice,

and affords a safety factor of three (on the elastic limit).

The following considerations indicate that the safety factor may be judiciously reduced as the span is increased:

1. On a longer span it takes more time for a maximum load to come upon the bridge, so that the application of stress is more gradual.

2. On a longer span with the same traffic, the combination of loads producing maximum stress will be much rarer in occurrence.

3. As the span increases, the dead-load becomes a greater percentage of the total load, so that the range of stress variation in the cable is diminished.

4. The resistance of tension members to suddenly applied stress increases with their length, on account of the increase in the resilience of the members.

5. It is reasonable to expect some improvement in the cable material before the larger spans are built. Furthermore, the best material can be afforded in the largest spans.

For the above reasons, the unit cable stress should be specified as an increasing function of the span. For this purpose, the following formula will be convenient:

$$s_0 = 180{,}000 \cdot \left(\frac{l+2000}{l+9000}\right); \quad . \quad (19)$$

this gives

$$\left.\begin{aligned} s_0 &= 60{,}000 \text{ at } l = 1500 \\ s_0 &= 75{,}000 \text{ at } l = 3000 \\ s_0 &= 86{,}500 \text{ at } l = 4500 \end{aligned}\right\} \quad . \quad (19')$$

and causes s_0 to approach the limiting value of $s_0 = 180.000 (= \text{E.L.})$ as l approaches ∞.

For substitution in the general formulæ of the preceding investigation, it will be more convenient to replace (19) by a linear equation which will give essentially the same values for s_0 for all practical values of l. Such an equation is

$$s_0 = 45{,}000 + 10l \quad . \quad . \quad . \quad (20)$$

This gives, as before,

$$\left.\begin{aligned} s_0 &= 60{,}000 \text{ at } l = 1500 \\ s_0 &= 75{,}000 \text{ at } l = 3000 \end{aligned}\right\} \quad . \quad (20')$$

Economic Rise-ratio, Corrected for Variable s_0. Instead of assuming the constant value of $s_0 = 60{,}000$, let the value of s_0 be specified by the linear formula

$$s_0 = 45{,}000 + 10l. \quad . \quad . \quad (20)$$

Substituting this value in Eqs. (18*a*) and (18*b*) we obtain the two formulæ

$$n = \frac{.0093l + \sqrt{.01359 + .0027l + .0000787l^2}}{1 + 0.198l} \qquad (21a)$$

and

$$n = \frac{.00874l + \sqrt{.01701 + .0033l + .000072l^2}}{1 + 0.192l} \qquad (21b)$$

Case I. If there are no suspenders in the side-spans, Eq. (21*a*) gives for

$l = 0$, $(s_0 = 45{,}000)$, $n = .116$
$l = 1000$, $(s_0 = 55{,}000)$, $n = .115$
$l = 2000$, $(s_0 = 65{,}000)$, $n = .113$
$l = 3000$, $(s_0 = 75{,}000)$, $n = .111$
$l = 4000$, $(s_0 = 85{,}000)$, $n = .110$

Hence, for suspension bridges of this type, the economic cable-rise is about *one-ninth* of the span.

Case II. If the side-spans are also suspended from the cable, Eq. (21*b*) gives for

$$\begin{aligned} l&=0, \quad (s_0=45{,}000),\ n=.130\\ l&=1000', \ (s_0=55{,}000),\ n=.127\\ l&=2000', \ (s_0=65{,}000),\ n=.124\\ l&=3000', \ (s_0=75{,}000),\ n=.122\\ l&=4000', \ (s_0=85{,}000),\ n=.120 \end{aligned}$$

Hence, for suspension bridges of this type, the economic cable-rise is about *one-eighth* of the span.

The above rise-ratios will be used in the following designs. The versed-sine (f_1) in the side-span is fixed by the relation

$$\frac{f_1}{f}=\left(\frac{l_1}{l}\right)^2 \quad . \quad . \quad . \quad (22)$$

necessary for equal cable inclinations at the towers. In the absence of any governing conditions, the side-spans will

be assumed one-half the length of the main span, so that we must have

$$f_1 = \frac{f}{4} \quad . \quad . \quad . \quad . \quad (22')$$

Art. 5

Minimum Depth for the Stiffening Truss

If a simple truss of span l is covered with a uniform load q, the deflection at the mid-point will be

$$N_0 = \frac{5}{384} \frac{ql^4}{EI}; \quad . \quad . \quad . \quad (1)$$

the inclination (or slope) at the ends of the span will be

$$\frac{dN}{dx} = \int_0^{\frac{l}{2}} \frac{M}{EI} \cdot dx = \frac{16}{384} \frac{ql^3}{EI}; \quad . \quad (2)$$

the bending moment at the mid-point is

$$M = \frac{ql^2}{8}; \quad . \quad . \quad . \quad . \quad . \quad (3)$$

and the corresponding chord-stress is

$$s_1 = \frac{Md}{2I}, \quad . \quad . \quad . \quad . \quad . \quad (4)$$

where d is the depth of truss and I is its moment of inertia (assumed constant).

Eliminating M and q from Eqs. (1), (3) and (4), we obtain

$$s_1 = 4.8\frac{Ed}{l^2} \cdot N_0. \quad . \quad . \quad (5)$$

Eliminating q from Eqs. (1) and (2) we obtain

$$\frac{dN}{dx} = 3.2\frac{N_0}{l}. \quad . \quad . \quad . \quad . \quad (6)$$

Eliminating N_0 from Eqs. (5) and (6), we find

$$\frac{dN}{dx} = \frac{2}{3} \cdot \frac{s_1}{E} \cdot \frac{l}{d}. \quad . \quad . \quad . \quad (7)$$

In the stiffening truss hinged at the towers, the maximum stresses occur with a load extending over one-half of the span. Each half of the truss then acts very nearly as a simple beam carry-

ing a uniform load (=actual load−suspender forces). We may therefore apply the preceding formulæ, (1) to (7), upon replacing l by the half-span of the suspension bridge $\left(=\frac{l}{2}\right)$. Eq. (7) then becomes

$$\frac{dN}{dx}=\frac{1}{3}\frac{s_1}{E}\cdot\frac{l}{d}. \quad . \quad . \quad . \quad (7')$$

Hence, if the maximum allowable grade $\left(\frac{dN}{dx}\right)$ is specified, the minimum depth of truss will be defined by

$$\frac{d}{l}=\frac{1}{3}\frac{s_1}{E}\div\frac{dN}{dx}. \quad . \quad . \quad (8)$$

If $s_1=20{,}000$ (i.e., 30,000 minus wind-stresses, secondary stresses, column-flexure stresses, etc.), $E=30{,}000{,}000$, and $\frac{dN}{dx}=1$ per cent (a limiting value for railroad bridges), we find

$$\text{Min.}\ \frac{d}{l}=\frac{1}{45}. \quad . \quad . \quad . \quad (9)$$

Art. 6

Economic Depth of Stiffening Truss

It can readily be shown that the cost of the cable is unaffected by any change in the depth of the stiffening truss, within the limits of practice. The effect upon the weight of the web-members is also negligible. We may therefore define the economic depth of stiffening truss as the depth which will render the chord-areas of the truss a minimum.

By Eq. (5) of the preceding article the intensity of stress in the chords of the truss produced by a cable deflection $= \Delta f$ is

$$s = 4.8\frac{Ed}{l^2} \cdot \Delta f. \quad . \quad . \quad . \quad (1)$$

If ΔL is the total elongation of the cable, then [1]

$$\Delta f = \frac{15}{16(5n - 24n^3)} \cdot \Delta L \quad . \quad . \quad (2)$$

[1] Cf. Melan, *Theorie der eisernen Bogen- und Hängebrücken* (Leipzig, 1906), p. 14, Eq. (39.)

For $n=\frac{f}{l}=\frac{1}{8}$, this equation reduces to

$$\Delta f = 1.62 \cdot \Delta L \quad . \quad . \quad . \quad (2')$$

The cable-stretch, ΔL, is composed of the elongation due to live-load plus the temperature expansion, or

$$\Delta L = \frac{s_0}{E} \cdot \frac{p}{g+p} \cdot L \pm wt \cdot L, \quad . \quad (3)$$

where s_0 is the total cable stress producible by the combined effect of the live-load (p) and the dead-load (g).

The total length of the cable is given by [1]

$$L = l \cdot \left(1+\frac{8}{3}n^2-\frac{32}{5}n^4\right) + 2l_1\left(1+\frac{8}{3}n_1^2\right)\sec \alpha_1 \quad (4)$$

For $n=\frac{1}{8}$ and $n_1=\frac{1}{16}$, this yields

$$L = 2.11l. \quad . \quad . \quad . \quad (4')$$

[1] Cf. Melan, *Theorie der eisernen Bogen- und Hängebrücken* (Leipzig, 1906), p. 14, Eq. (43).

Substituting the values of (2′), (3) and (4′) successively in (1), we obtain

$$s = 16.44\left(s_0\frac{p}{g+p} + Ewt\right)\frac{d}{l} \quad . \quad (5)$$

If M is the bending moment produced by the live-load, and k is the corresponding unit stress, the chord-section will be

$$A_1 = \frac{M}{k.a} \quad . \quad . \quad . \quad . \quad . \quad (6)$$

The value of the moment may be written

$$M = m \cdot p \cdot l^2, \quad . \quad . \quad . \quad (7)$$

where m is a factor nearly constant for all spans

$$(= \tfrac{1}{10} \text{ approximately } [1]).$$

If s_1 is the total allowable chord-stress, deducting the stress (s) caused by the deflections, there remains,

$$k = s_1 - s \quad . \quad . \quad . \quad . \quad (8)$$

[1] Cf. Burr, *Ancient and Modern Engineering* (1903), p. 175.

Substituting the values of (7), (8) and (5) successively in (6), there results

$$A_1 = \frac{mpl^2}{d\left\{ s_1 - 16.44\left(s_0\frac{p}{g+p} + Ewt\right)\frac{d}{l}\right\}} \quad (9)$$

For a minimum chord-area, the denominator of the above expression must be made a maximum. The necessary condition is

$$s_1 - 32.88\left(s_0 \cdot \frac{p}{g+p} + Ewt\right)\frac{d}{l} = 0,$$

or

$$\text{Economic } \frac{d}{l} = \frac{s_1}{32.88\left(s_0\frac{p}{g+p} + Ewt\right)} \quad (10)$$

Substituting

$$s_1\left(= \frac{30000}{1 + \frac{l^2}{8000r^2}}\right) = 26000,$$

and $Ewt = 12000$, Eq. (10) becomes

$$\text{Economic } \frac{d}{l} = \frac{791}{s_0\frac{p}{g+p} + 12000}. \quad (10')$$

Assuming a cable-stress increasing with the span-length (cf. Art. 4) and taking $\frac{g}{p}$ from the results of the writer's designs, we find, with the aid of Eq. (10′), for

$l=1500$, $\frac{g}{p}=1.8$, $s_0=60{,}000$, Econ. $\frac{d}{l}=\frac{1}{42}$;

$=2250$, $=2.3$, $=67{,}500$, $=\frac{1}{41}$;

$=3000$, $=3.1$ $=75{,}000$ $=\frac{1}{38}$.

Mean $=\frac{1}{40}$.

This value is somewhat higher than the average of past practice, probably because most designs have been a compromise between the demands of economy and those of æsthetics. The Williamsburg Bridge is the only long-span structure conforming to the above economic ratio, but its appearance is undoubtedly marred by the excessive depth of the stiffening truss.

Eq. (9) yields

$$A_1 \doteq \infty \text{ for } \begin{cases} \dfrac{d}{l} \doteq 0, \text{ or,} \\ \dfrac{d}{l} \doteq \dfrac{s_1}{16.44\left(s_0 \dfrac{p}{g+p} + Ewt\right)} \\ \qquad = 2 \cdot \left(\text{Econ. } \dfrac{d}{l}\right). \end{cases}$$

That is, the cost of the stiffening truss approaches infinity as the depth departs from the economic value either toward a value twice as great or toward a zero value. This shows that the truss depth is a very important factor in the economic design of a suspension bridge.

For the purpose of this investigation, economy of weight and cost is of greater significance than any æsthetic considerations. We shall therefore adhere to the depth-ratio derived above, even at a sacrifice of appearance.

It will be noted that the above value of $d\left(=\frac{1}{40}l\right)$ is but little in excess of the

minimum permissible depth $\left(=\frac{1}{45}l\right)$ as found in the preceding section.

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CHAPTER III

DESIGN OF SUSPENSION BRIDGES

ART. 7

PRINCIPAL DATA FOR DESIGN OF SUSPENSION BRIDGES

	No. 1.	No. 2.	No. 3.
l = Span	= 1500,	2250,	3000 ft.
l_1 = Side-span = $l/2$	= 750,	1125,	1500 ft.
f = versed-sine = $0.12l$	= 180,	270,	360 ft.
f_1 = versed-sine in side-span = $f/4$	= 45,	67.5,	90 ft.
d = depth of truss = $.024l$	= 36,	54,	72 ft.
a = panel length (between suspenders)	= 18.5,	22.3,	24.8 ft.

Type: Stiffening truss hinged at the towers. Suspension rods in side-spans.

Loading:

4 railroad tracks at	3000 lbs.	= 12000 lbs. p.l.f.
2 lines of cars at	1000	= 2000
40 ft. of roadway at	75	= 3000
20 ft. of roadway at	50	= 1000
Total congested load		= 18000 lbs. p.l.f.

Total width of structure = 115 ft. + 2 cantilevers at 10 ft.

4 Cables, spaced 37'.5 + 40' + 37'.5 c. to c.

4 Trusses, spaced 37'.5 + 40' + 37'.5 c. to c.

Art. 8

Design of the Stiffening Truss

By proper adjustment of the suspension rods after the erection is completed, all strain may be taken out of the stiffening truss before live-load is applied. The dead-load is thus carried wholly by the cable and may be entirely omitted from consideration in designing the stiffening truss. The latter will therefore be designed merely for the uniform live-load of 4500 lbs. per linear foot, a temperature variation of $\pm 60°$ F., and a lateral wind pressure of 30 lbs. per square foot.

The formulæ used will be those based on the Theorem of Least Work, as developed by J. Melan in his "Theorie der eisernen Bogenbrücken und Hängebrücken." [1] This theory has been found

[1] *Handbuch der Ingenieur - Wissenschaften* (Leipzig, 1906), II. Band. V. Abteilung. XII. Kapitel, pp. 17–50. Cf. Müller-Breslau, "Theorie der durch einen Balken versteiften Kette." *Zeitschr. d. Arch. und Ing. Ver. zu Hannover*, 1881, p. 57. Am Ende, "Suspension Bridges with Stiffening Girders," Proc. of the Inst. of

by the writer to give results essentially the same as those yielded by formulæ established in a different manner by M. Maurice Lévy in his "Calcul des ponts suspendus rigides,"[1] and by those of C. Schwend in his treatise on suspension bridges.[2]

The above theories involve the following common assumptions:[3]

1. The cable is supposed perfectly flexible, freely assuming the form of the equilibrium polygon of the suspender forces.

2. The truss is considered a beam, initially straight and horizontal, of constant moment of inertia and tied to the cable throughout its length.

C.E., 1899, p. 306; and A. J. Dubois, "The Stresses in Framed Structures," New York, 1896.

[1] *Annales des Ponts et Chausseés*, 1886, II., p. 179, et seq.

[2] Schwend, *Ueber Berechnung und Konstruktion von Hängebrücken unter Anwendung von Stahldraht-Kabeln und Versteifungsbalken* (Leipzig, 1887).

[3] Cf. M. T. Godard, *Annales des Ponts et Chausseés*, 1894, II., and Melan, p. 17.

3. The dead-load of truss and cable is assumed uniform per linear unit, so that the initial curve of the cable is a parabola.

4. The form and ordinates of the cable curve are assumed to remain unaltered upon application of loading.

All of these assumptions, with the exception of the last, are very near the actual conditions even in the case of flexible trusses. The last assumption is admissible only with trusses of sufficient rigidity, and its exclusion results in the "Genauere Theorie," developed by Melan.[1] For the purposes of this investigation, the Least Work Theory will be deemed sufficiently accurate, its variations from the Exact Theory being negligible and on the side of safety.

Live-load Stresses. The bending moment at any section (x) of the main span (l), produced by a load (p) covering the entire length of the bridge, is

$$M_{tot}=\tfrac{1}{2}px(l-x)\left[1-\frac{8}{5N}\left(1+2\frac{f_1}{f}\cdot\frac{I}{I_1}\cdot\frac{{l_1}^3}{l^3}\right)\right] \quad (1)$$

[1] Melan, p. 50, et seq.

where

$$N=\tfrac{8}{5}\left(1+2\frac{I}{I_1}\cdot\frac{l_1}{l}\cdot\frac{f_1^2}{f^2}\right)+3\frac{I}{A_0f^2}$$
$$\cdot\left[1+\frac{16}{3}\cdot\frac{f^2}{l^2}\cdot+2\frac{l_2}{l}\left(1+\frac{16}{3}\frac{f_1^2}{l_1^2}+\tan^2\alpha_1\right)\right] \quad (2)$$

The results of preliminary estimates of truss and cable gave

for $l = 1500 \quad 2250 \quad 3000$

$$\frac{I}{A_0f^2}= \frac{1}{62} \quad \frac{1}{75} \quad \frac{1}{94}.$$

With these values, assuming $I=I_1$, we obtain

$$N = 1.82, \quad 1.80, \quad 1.78.$$

and

$$M=0.0322, \quad 0.0278, \quad 0.0230px(l-x).$$

The same relation obtains for the side spans, upon substituting l_1 for l. The above figures may be interpreted as indicating that the truss in each case serves to carry

$$6.44\% \qquad 5.56\% \qquad 4.60\%$$

of the total load, the remainder being carried by the cable.

The position ($\xi = n \cdot l$) of a concentration producing zero bending moment at any section (x) of the truss is called the *critical point* for that section and is given by

$$-n^3+n^2+n = N \cdot \frac{f}{l} \cdot \frac{x}{y}$$

$$= .218, \quad .216, \quad .214\frac{x}{y}. \qquad (3)$$

The roots of this equation for different values of x are most easily obtained from a graph of the function

$$f(n) = -n^3+n^2+n,$$

which, once plotted, can be used for all suspension bridge designs.

For the greatest negative moment at any section (x) of the main span, the load should cover both side spans and the portion ($l-\xi$) of the main span between the farther end and the critical point. The resulting bending moment is

$$M_{\min} = -\frac{2px(l-x)}{5N}$$

$$\left\{[2-n-4n^2+3n^3](1-n)^2+4\frac{I}{I_1}\cdot\frac{l_1^3}{l^3}\cdot\frac{f_1}{f}\right\} \qquad (4)$$

A graph of the function $f(n) = (2 - n - 4n^2 + 3n^3)$ was used by the writer to simplify the labor involved in the repeated application of the above formula.

For the greatest positive moment at any section of the main span, that span is loaded from $x = 0$ to $x = \xi$, giving

$$M_{max} = M_{tot} - M_{min}. \quad . \quad (5)$$

There are no critical points in the side spans. For the greatest negative moment at any section (x) in one of the side spans, load the other two spans, giving

$$M_{min} = -\frac{2px(l_1 - x)}{5N} \left\{ 2\frac{f_1}{f} \cdot \frac{l^2}{l_1^2}\left(1 + \frac{I}{I_1}\frac{l_1^3}{l^3} \cdot \frac{f_1}{f}\right) \right\} \quad (6)$$

which reduces to

$$M_{min} = -.455, \quad -.459, \quad -.462px(l_1 - x).$$

Loading the span itself produces the greatest positive moments:

$$M_{max} = M_{tot} - M_{min} = .487, \ .487, \ .486px(l_1 - x).$$

With all spans completely loaded, the shear at any section (x) of the main span will be

$$S_{tot}=\tfrac{1}{2}p(l-2x)\left[1-\frac{8}{5N}\left(1+2\frac{I}{I_1}\cdot\frac{l_1^3}{l^3}\cdot\frac{f_1}{f}\right)\right] \quad (7)$$

In the designs at hand, this reduces to

$$S_{tot}=.0644,\quad .0556,\quad .0460p\left(\frac{l}{2}-x\right),$$

the coefficient in each case representing the fraction of the total load carried by the truss. The same relations apply to the side spans upon replacing l by l_1.

The critical point ($\xi=n\cdot l$) for zero shear at any section (x) of the truss is given by

$$n+n^2-n^3=\frac{N}{4}\cdot\frac{l}{l-2x}. \quad (8)$$

This formula is solved by the same graph as that used for Eq. (3). For maximum shear at any section of the truss, the load should extend from $x=0$ to the given section, and from the critical

point to the farther end of the span. The shear will then be

$$S_{max} = \tfrac{1}{2}pl\left(1-\frac{x}{l}\right)^2 \cdot\left\{1-\frac{8}{N}\left(1-\frac{2x}{l}\left(\left[\frac{1}{5}\left(1-\frac{x}{l}\right)^3-\frac{1}{2}\left(1-\frac{x}{l}\right)^2+\frac{1}{2}\right]\right.\right.\right\} + \tfrac{1}{2}pl(1-n)^2 \cdot\left\{\frac{8}{N}\left(1-\frac{2x}{l}\right)[\tfrac{1}{5}(1-n)^3-\tfrac{1}{2}(1-n)^2+\tfrac{1}{2}]-1\right\} \quad (9)$$

There will be no critical point for any section where

$$x > \frac{l}{2}\left(1-\frac{N}{4}\right) = .272l, \quad .275l, \quad .278l.$$

For all such sections, $n=1$, and the last term in the equation for S_{max} vanishes.

There are no critical points in the side spans, the maximum shear at any section being simply

$$S_{max} = \tfrac{1}{2}pl_1\left(1-\frac{x}{l_1}\right)^2 \cdot\left\{1-\frac{8}{N}\cdot\frac{I}{I_1}\cdot\frac{l_1}{l}\cdot\frac{f_1^2}{f^2}\left(1-\frac{2x}{l_1}\right)\left[\frac{1}{5}\left(1-\frac{x}{l_1}\right)^3-\frac{1}{2}\left(1-\frac{x}{l_1}\right)^2+\frac{1}{2}\right]\right\} \quad (10)$$

A graph of the function

$$f(n) = \tfrac{1}{6}(1-n)^3 - \tfrac{1}{2}(1-n)^2 + \tfrac{1}{2}$$

serves to simplify the application of Eqs. (9) and (10).

The stresses in the diagonal web members are obtained by dividing the shear at any section by the number of web members cut by that section, and multiplying the quotient by the secant of their inclination to the vertical.

Temperature Stresses. The cable-tension produced by a rise in temperature (t) is given by

$$H_t = -\frac{3EIwtL}{f^2N \cdot l} \quad . \quad . \quad . \quad (1)$$

For an extreme variation of $t = \pm 60°$ F., $Ewt = 11{,}720$. Taking the value of I from preliminary estimates, Eq. (1) yielded

$$H_t = 260, \quad 420, \quad 560 \text{ kilo-pounds.}$$

The resulting bending moment at any section of the truss is given by

$$M_t = H_t Y \quad . \quad . \quad . \quad . \quad (2)$$

and the transverse shear by

$$S_t = H_t \cdot \tan \tau, \quad . \quad . \quad (3)$$

where τ is the inclination of the cable at the given section.

Wind Stresses. Let $p =$ the total horizontal wind-load per linear foot. The resulting lateral deflection (h) causes a displacement of the plane of the cable and suspenders from the vertical, thereby giving rise to a force of restitution (r) equal to the horizontal component of the suspender tensions. If v is the mean vertical distance of the truss and live load below the cable chord, then the reaction component of the displaced weight (W) will be

$$r = \frac{h}{v} \cdot W. \quad . \quad . \quad . \quad . \quad (1)$$

Considering the truss as a beam acted on by a uniform horizontal load $(p - r)$, we have

$$h = \frac{5}{384}(p - r)\frac{l_4}{EI}. \quad . \quad . \quad (2)$$

From (1) and (2), there results

$$r = \frac{\frac{W}{v} \cdot \frac{5}{384} \cdot \frac{l_4}{EI}}{1 + \frac{W}{v} \cdot \frac{5}{384} \cdot \frac{l_4}{EI}} \cdot p. \quad . \quad (3)$$

In the designs at hand, Eq. (3) yields

$$r/p = 19\%; \quad 45\%; \quad 57\%$$

in the main spans, and

$$r/p = 3.5\%; \quad 9.3\%; \quad 20.3\%$$

in the side-spans.

The specified wind-load (p) consists of a pressure of 30 lbs. per square foot acting on the exposed surface of one truss and on half that of the remaining trusses, also on a train of cars 14 ft. high. On the round surface of the cables, only one-half of the above pressure is considered effective. The total wind pressure in our designs is thus found to be

$$p = 1220; \quad 1400; \quad 1545 \text{ lbs. p.l.f.}$$

and the *effective* horizontal pressure will be

$$p - r = 988; \quad 772; \quad 671 \text{ lbs. p.l.f.}$$

in the main spans, and

$$p-r = 1177; \quad 1270; \quad 1231 \text{ lbs. p.l.f.}$$

in the side-spans.

The resulting lateral deflections, by Eq. (2), will then be

$$h = 18\tfrac{1}{2}''; \quad 48\tfrac{1}{4}''; \quad 61''$$

at the center of the main span, and

$$h = 1\tfrac{3}{8}''; \quad 3\tfrac{1}{4}''; \quad 7''$$

at the center of the side-span.

Each of the above pressures $(p-r)$ must be resolved into two parts acting in the planes of the upper and lower lateral systems respectively. On account of the wind pressure on the floor and train, the lower chords get the major share of the wind-load, amounting, in the present designs, to about 63% of the total pressure.

The system of bracing adopted is similar to that used on the Manhattan Bridge. The upper lateral bracing consists of diagonals and struts connecting the inside to the outside trusses. There is no bracing over the roadway or sidewalks. The

upper lateral system is therefore composed of two independent horizontal trussses, 37.5 ft. deep, so that the chord-stresses due to the wind moments will equal $M_w \div 75$.

The lower lateral system consists of the four lower chords tied together by the floor beams and special diagonal braces extending across the entire floor. Assuming the wind-stresses in the chords to vary as their respective distances (20 ft. and 57.5 ft.) from the neutral axis, the stress in the outside lower chords will equal $M_w \div 129$.

It is thus found that the wind-stresses in the upper and lower chords are almost equal, so that the same sections may be used for both.

In designing the members of the lateral systems, the wind pressure is considered as a moving load. The maximum shear at any section (x) is given by

$$S = \frac{p}{2l}(l - x)^2,$$

and the resulting stresses are obtained

by dividing this shear by the number of laterals cut by the section and multiplying the quotient by the secant of their inclination.

Working Stresses. The chords of the stiffening truss are made of nickel steel and are designed for working stresses of 30,000 lbs. per square inch in tension and

$$\text{for } 30{,}000 \div \left(1 + \frac{l^2}{8000r^2}\right) \text{ in compression.}$$

The web members, bracing and connection details are of structural steel and are designed for working stresses of 20,000 lbs. per square inch in tension and

$$\text{for } 20{,}000 \div \left(1 + \frac{l^2}{8000r^2}\right) \text{ in compression.}$$

Computations. The limitation of space renders it impracticable to reproduce here the detailed computations of the stresses and weights of members. For the same reason, the design of the floor system will not be given here. The final weights, however, are tabulated in the following article:

Art. 9

Design of Suspenders

The load carried by the suspenders consists of the following items:

	S. B. No. 1.	S. B. No. 2.	S. B. No. 3.	
Truss and bracing	2,624	4,015	5,786	lbs.
Floor	3,712	3,800	3,812	
Suspenders	126	207	320	
Total Dead-load	6,462	8,022	9,918	
Live-load	4,500	4,500	4,500	
Total load p.l.f. ($=g_1$)	10,962	12,522	14,418	lbs.
Total load per suspender	202,800	279,800	358,000	lbs.

With a specified working stress of 30,000 lbs. per square inch, the required sections for the suspenders will be:

	6.8	9.3	11.9	sq. in.
Their average length ($=\frac{1}{3}f+d$) =	96	144	192	ft.
Hence, their mean weight =	126	207	320	

lbs. p.l.f. of truss.

as assumed above.

Art. 10

Design of Cables

Preliminary Estimate. For an accurate design of the cable its weight must be known in advance. For this purpose, use is made of Eq. (5) of Art. 4, viz.:

$$g_0 = g_1 \cdot \frac{k \cdot l \cdot \frac{1}{n}\sqrt{1+16n^2}}{1 - kl \cdot \frac{1}{n}\sqrt{1+16n^2}}.$$

With $k = \frac{1}{136000}$, and $n = \frac{f}{l} = 0.12$, the above reduces to

$$g_0 = g_1 \frac{.000068l}{1 - .000068l},$$

or, for

$$l = 1500, \quad 2250, \quad 3000,$$
$$g_0 = .114g_1, \quad .181g_1, \quad .257g_1.$$

Taking the values of the total suspended load (g_1) from the preceding article, we find the weight of the cable to be

$$g_0 = 1250, \ 2270, \ 3710 \text{ lbs. p.l.f.}$$

After a first design these values were slightly altered to those given below for the final computation.

Final Computation. The total load carried by the cables consists of the following items:

	S. B. No. 1.	S. B. No. 2.	S. B. No. 3.
Dead-load on suspenders (as above)	6,462	8,022	9,918
Weight of cable	1,286	2,160	3,720
Total dead-load ($=g$)	7,748	10,182	13,638
Live-load ($=p$)	4,500	4,500	4,500
Total load p.l.f. ($=g+p$)	12,248	14,682	18,138

For a live-load (p) covering the main span, the horizontal component of the cable tension will be

$$H=\frac{1}{5N}\frac{pl^2}{f}. \quad . \quad . \quad . \quad (1)$$

In the designs at hand, this gives

$$H=.9170;\quad .9250;\quad .9365\ pl.$$

For a live-load (p) covering one of the side-spans, the cable tension is given by

$$H=\frac{1}{5N}\frac{f_1}{f}\cdot\frac{I}{I_1}\cdot\frac{l_1^2}{l^2}\cdot\frac{l}{f}\cdot pl_1. \quad . \quad (2)$$

In the present designs this gives

$$H = .0572;\quad .0578;\quad .0585\ pl_1.$$

Combining the above values, we find the cable tension for a uniform live-load (p) covering all the spans to be

$$H_p = .9742;\quad .9840;\quad .9950\ pl.$$

As the cable sustains the entire dead-load (g) without any relief from the stiffening truss, the corresponding horizontal tension will be

$$H_g = \frac{gl^2}{8f} = \frac{gl_1^2}{8f_1}, \quad . \quad . \quad . \quad (3)$$

or, in the designs at hand,

$$H_g = 1.0417\ gl.$$

Substituting the values of (g) and (p) in the above expressions for H, we obtain

H_g =	12,100,000,	23,880,000,	42,600,000 lbs.
H_p =	6,135,000,	9,960,000,	13,440,000 lbs.

As previously given (Art. 8), the cable tension producible by a fall of temperature is

H_t =	260,000,	420,000,	560,000 lbs.

Hence,

Total H=18,495,000, 34,260,000, 56,600,000 lbs.

Multiplying this value by the secant of the inclination of the cable at the towers, (sec.=1.109), we find the maximum cable tension to be

T_{max}=20,510,000 lbs., 38.000,000 lbs., 62,770,000 lbs.

With the specified unit stress of 60,000 lbs. per square inch, the required cable section (per truss) will be

$$A_0=342,\quad 633,\quad 1046 \text{ sq.in.}$$

This section will be provided as follows:

S. B. No. 1. One cable of 37 strands, each containing 319 wires of 0.192 in. diam. Diam.=23¾ in. Total area=342 sq.in.

S. B. No. 2. Two cables of 37 strands, each containing 296 wires of 0.192 in. diam. Diam.=22⅞ in. Total area=634 sq.in.

S. B. No. 3. Three cables of 37 strands, each containing 326 wires of 0.192 in. diam. Diam.=24 in. Total area=1047 sq.in.

Each cable will be wrapped with a single layer of No. 10 (B. W. G.) iron wire.

Allowing for catenary, cable wrapping, etc., the mean weight of the cables will be

$$g_0 = 1286,\quad 2160,\quad 3720 \text{ lbs. p.l.f.}$$

as assumed above.

Art. 11

Design of Towers

Each tower will consist of four box-columns, one for each cable system, rigidly tied together by transverse bracing in a vertical plane. Comparative designs by the writer indicated a small economy of material in favor of pin-bearing columns, but this saving is more than outweighed by the more expensive construction, greater difficulty of erection, uncertainty of action and æsthetic inferiority due to the impression of instability. The more

usual design, namely with the columns rigidly fixed at the base, will therefore be followed.

Assuming the required clearance of the truss above M. H. W. to be 135, 160 and 185 ft. for the respective bridges, and assuming the top of the masonry piers to be 30 ft. above M. H. W., the total height of the steel towers will be: for

S. B. No. 1, $180+36+135-30=321$ ft.
S. B. No. 2, $270+54+160-30=454$ ft.
S. B. No. 3, $360+72+185-30=587$ ft.

Deducting the height of the pedestal castings, the effective height of the towers will be

$$h=316;\ 448;\ 580 \text{ ft.};$$

and the height up to the stiffening truss will be

$$h_1=100;\ 124;\ 148 \text{ ft.}$$

The maximum fiber stress in the tower columns will occur when the live-load covers the main span and the farther

side-span at maximum temperature. Under this condition of loading, the top of the tower will be deflected toward the main span as a result of the following deformations:

1. The upward deflection (Δf_1) at the center of the unloaded side-span.

2. The elongation of the cable between the anchorage and the tower due to the elastic strain produced by the applied loads.

3. The elongation of the cable due to thermal expansion.

These deformations are computed as follows:

(1) The upward deflection (Δf_1) is found by considering the side-span as a simple beam subjected to a downward loading equal to the suspended dead-load (p) and an upward loading equal to the suspender tensions $\left(t = H \div \frac{l^2}{8f}\right)$. If $p_1 (= t - p)$ is the resultant of these loadings, the central deflection will be

$$\Delta f_1 = \frac{5}{384} \frac{p_1 l_1^4}{EI} \quad . \quad . \quad . \quad (1)$$

For the spans under investigation, this gives

$$\Delta f_1 = 2.630;\ 3.760;\ 4.885 \text{ ft.}$$

(2) The elastic elongation of the cable in the side-span is given by

$$\Delta L_1 = \frac{H}{EA_0} \cdot \int \frac{ds^3}{dx^2}$$

$$= \frac{H}{EA_0} \cdot l_1 \left(1 + 8\frac{f_1^2}{l_1^2} + \frac{3}{2}\tan^2 \alpha_1\right), \quad (2)$$

which reduces to

$$\Delta L_1 = 1.915;\ 2.875;\ 3.860 \text{ ft.}$$

(3) The temperature expansion of the cable in the side-span is given by

$$\Delta L_1 = wt \int \frac{ds^2}{dx}$$

$$= wt \cdot l_1 \left(1 + \frac{16}{3}\frac{f_1^2}{l_1^2} + \tan^2 \alpha_1\right), \quad (3)$$

which reduces to

$$\Delta L_1 = 0.390;\ 0.585;\ 0.780 \text{ ft.}$$

The length of the cable in the side-span is given very closely by

$$L_1 = l_1 \left(1 + \frac{8}{3}\frac{f_1^2}{l_1^2} + \frac{1}{2}\tan^2 \alpha_1\right), \quad (4)$$

from which we find

$$\frac{dL_1}{dl_1}=\left(1+\frac{8}{3}\frac{f_1^2}{l_1^2}+\tfrac{1}{2}\tan^2\alpha_1\right)=1.125, \quad (5)$$

and

$$\frac{dL_1}{df_1}=\frac{16}{3}\frac{f_1}{l_1}=0.32. \quad . \quad . \quad (6)$$

The deflection of the top of the tower is then given by

$$y_0=\Delta l_1=\frac{dl_1}{dL_1}\Sigma(\Delta L_1)+\frac{dl_1}{dL_1}\frac{dL_1}{df_1}\cdot\Delta f_1. \quad (7)$$

Substituting the values from (1), (2), (3), (5) and (6) in (7), we obtain the maximum tower deflection,

$$y_0=2.798; \quad 4.150; \quad 5.515 \text{ ft.}$$

Considering this deflection as produced by an unbalanced horizontal force P applied at the top of the tower, this force may be calculated if the sectional dimensions of the tower are known. As these are not known in advance, the following procedure is adopted:

Assume the moment of inertia of the section of the tower to have a maximum value (I_0) at the base and to diminish regularly toward the top according to some appropriate law. A study of actual designs indicates the applicability of the law of variation represented by the empirical formula

$$I = I_0(1 - \sqrt{x/h}), \quad . \quad . \quad (8)$$

where x is the distance of any section above the base. Substituting this relation in the differential equation of the elastic curve of the cantilever

$$EI\frac{d^2y}{dx^2} = P(h - x),$$

and integrating, there results

$$P = \frac{30}{23}\frac{EI_0}{h^3}y_0. \quad . \quad . \quad . \quad (9)$$

In the designs at hand, this reduces to

$$P = 3.470I_0; \quad 1.736I_0; \quad 1.094I_0.$$

The other loads acting on the tower are the vertical reaction ($V = 2H \cdot \tan \alpha$)

at the saddles, and the end-shears (V_1) at the point of suspension of the stiffening truss. In the designs at hand, for the condition of loading under consideration, we find

$V =$ 17,150,000; 30,700,000; 52,100,000 lbs.
$V_1 = -1{,}037{,}000; \ -1{,}852{,}000; \ -2{,}640{,}000$ lbs.

At any section (x) of the tower, the horizontal deflection (y) from the initial vertical position of the axis is given with sufficient accuracy by the equation for the elastic curve of the cantilever:

$$y = y_0\left[\frac{3}{2}\left(\frac{x}{h}\right)^2 - \frac{1}{2}\left(\frac{x}{h}\right)^3\right]. \quad . \quad (10)$$

This gives, for $x = h_1$,

$$y_1 = 0.376, \ 0.436, \ 0.492 \text{ ft.}$$

The maximum fiber stress at any section of the tower will be

$$s = \frac{V}{A} + \frac{M \cdot d}{I} + \left[\frac{V_1}{A}\right]_0^{h_1}, \quad . \quad (11)$$

where

$$M = P(h-x) + V(y_0 - y) + [V_1(y_1 - y)]_0^{h_1}. \quad (12)$$

The moment of inertia for the form of section used here is given approximately by

$$I = 0.50A \cdot d^2. \quad . \quad . \quad . \quad (13)$$

Substituting (12) and (13) in (11), and applying the resulting equation to the base of the tower, we find

$$s = \left(\frac{P}{I_0}\right) \cdot hd_0 + \frac{2Vy_0}{A_0 d_0} + \frac{2V_1 y_1}{A_0 d_0} + \frac{V + V_1}{A_0}. \quad (14)$$

Using an allowable fiber stress of $s = 24{,}000$ lbs. per square inch, and substituting the numerical values of P/I_0, V, V_1, h, y_0, and y_1, as given above, there results the following relation:

S. B. No. 1:

$$24{,}000 = 1096d_0 + \frac{96000000}{A_0 d_0} + \frac{16113000}{A_0},$$

S. B. No. 2:

$$24{,}000 = 788d_0 + \frac{255000000}{A_0 d_0} + \frac{28848000}{A_0},$$

S. B. No. 3:

$$24{,}000 = 647d_0 + \frac{800000000}{A_0 d_0} + \frac{49460000}{A_0}.$$

This relation is satisfied by the following values:

S. B. No. 1: $A_0 = 3240$ sq.in.,
$d_0 = 15$ ft.; $\therefore I_0 = 365{,}000$ in.^{2} ft.^{2}

S. B. No. 2: $A_0 = 4260$ sq.in.,
$d_0 = 17$ ft.; $\therefore I_0 = 625{,}000$ in.^{2} ft.^{2}

S. B. No. 3: $A_0 = 7200$ sq.in.,
$d_0 = 20$ ft.; $\therefore I_0 = 1{,}440{,}000$ in.^{2} ft.^{2}

The horizontal force P is now determined from Eq. (9) as

$P = 694{,}000;\ 1{,}085{,}000;\ 1{,}575{,}000$ lbs.

All the remaining sections of the tower may now be proportioned, using Eq. (8) as a guide and checking the maximum fiber stress by Eqs. (11) and (12). The mean section of the tower is thus found to be

Mean $A = 2210;\ 3240;\ 5550$ sq.in.,

and the total weight, including connection details and bracing (about 100 per cent), is found to be

Weight $= 4{,}465{,}000;\ 9{,}720{,}000;\ 22{,}406{,}000$ lbs. per column. (Structural steel.)

Pedestals. The pressure at the foot of each tower column is a maximum when the live-load extends over all the spans, and is then made up of the following items:

	S. B. No. 1.	S. B. No. 2.	S. B. No. 3.
Max. Cable Reaction (V)	17,500,000	32,500,000	53,830,000
Max. Truss Reaction (V_1)	326,000	422,000	466,000
Wt. of Tower	4,465,000	9,720,000	22,406,000
Wt. of Pedestal	420,000	810,000	1,200,000
Total load (lbs.)	22,711,000	42,452,000	77,902,000
Bearing area required at 15 tons per sq. ft.	757	1448	2597

In order to evenly distribute the above loads over the requisite area of masonry, the tower legs will rest on pedestals of cast steel (annealed) having the following dimensions:

	S. B. No. 1.	S. B. No. 2.	S. B. No. 3.
Top of casting....	10×30	18×34	24×40
Bottom of casting	20×40	30×48	36×72
Height.....	5 ft.	6 ft.	7 ft.
Weight of casting (lbs.).....	420,000	810,000	1,200,000

Art. 12

Design of Masonry Piers

The steel castings at the bases of the tower columns are anchored on pedestal blocks of selected granite. The top of the pier is made just large enough to hold these pedestals, except where a larger section is required to reduce the pressure in the masonry to 12 tons per square foot. The pier is built of 1:2½:5 concrete with limestone (ashlar) facing. The upper pier, above the starling, is given the section of a rectangle with semicircular ends and has a batter of 1:20 on sides and ends. In the lower pier, the sides (batter 1:20) are continued at each end in two circular arcs to form a cutwater (batter 3:20).

Below the masonry is a rectangular cribwork, used as a coffer-dam during construction. It is built of 12×12 in. timber, and is filled with concrete as

the sinking progresses. It rests directly on the pneumatic caisson which has a steel working chamber 7 ft. high, and a reinforced-concrete roof 3 ft. thick. After reaching rock, the caisson and shafts are carefully filled with 1:2:4 Portland cement concrete.

Since the economic depth of foundations increases with the length of span, it will be assumed that the pier-depths for the bridges under design are somewhat greater for the longer spans. The following are the principal elevations assumed:

	S. B. No. 1.	S. B. No. 2.	S. B. No. 3.
Top of pier	30	30	30
Starling	5	5	5
Mean high water (=datum)	0	0	0
Base of pier=top of cribwork	−15	−15	−15
River bottom	−20	−25	−30
Base of cribwork=top of caisson	−80	−90	−100
Cutting-edge of caisson=surface of rock	−90	−100	−110

The principal sectional dimensions of the piers, determined as outlined above, are as follows:

	S. B. No. 1.	S. B. No. 2.	S. B. No. 3.
Top of pier.....	177×42	195×50	225×74
Base of upper pier.	179×44	197×52	227×76
Top of lower pier.	200×44	222×52	259×76
Base of pier.....	206×46	228×54	265×78
Cribwork.....	212×52	234×60	271×82
Caisson.....	212×52	234×60	271×82

A complete design and estimate of the piers yielded the following quantities of material required:

	S. B. No. 1.	S. B. No. 2.	S. B. No. 3.
Pier masonry.....	12,890	15,490	26,300 cu.yds.
Concrete filling in cribwork.....	25,300	37,400	67,800 "
Concrete filling in caisson.....	4,080	5,200	8,230 "
Timber in cribwork	620	790	1,080 {M.ft. B.M.
Steel in caisson and shafts.....	1,150,000	2,320,000	2,650,000 lbs.
Earth excavation..	28,600	39,000	65,000 cu.yds

Art. 13

Design of Anchorages

The anchorage, as a mass, is required to offer sufficient frictional resistance to sliding to resist the tension of the cable. Dividing the maximum horizontal tension (H) by the coefficient of friction ($\mu=0.6$),

the quotient will be the necessary weight of anchorage masonry. Introducing a factor of safety of 2, the masonry required for the anchorage is thus found to be

63,500; 109,000; 183,200 cu.yds.

This will be built of concrete with limestone facing as in the case of the piers.

Enclosed in the base of the anchorage are heavy box girders to which the cables are anchored by means of chains of eye-bars. The total amount of steel work in each anchorage is found to be

1,037,500; 2,472,500; 4,090,000 lbs.

The anchorage will be supported on a concrete foundation resting on bearing piles if necessary. The volume of material in this foundation is found to be

5,290; 10,400; 16,650 cu.yds.

Art. 14

Estimate of Cost

In order to establish an equitable schedule of unit prices for the two types of bridges, a critical study was made of the different bids received by New York City for the Williamsburg, Manhattan and Queensboro Bridges. These structures were selected as combining the qualifications of recentness of date with similarity of construction and magnitude to the designs under investigation. The prices of the successful bidders were adopted except where they differed so widely from the following bids as to indicate their having been unbalanced for some special reason. The unit prices determined by this comparison are incorporated in the following estimates of cost.

SUSPENSION BRIDGE. SPAN = 1500 FT.

Estimate of Cost.

Material.	Quantity.	Price.	Amount.	Total.
	lbs.	c.	$	$
Cables, etc.:				
4 cables at 342 sq. in.	15,940,000	12.5	1,992,500	
Suspenders	1,452,000	10.0	145,200	
Steel castings (cable bands, etc.)	1,645,000	9.0	148,050	
				2,285,750
Suspended superstructure:				
Truss chords (nickel steel)	23,632,000	8.0	1,890,560	
Webs and bracing (struct. steel)	7,269,000	5.6	407,060	
Floor system (structural steel)	16,026,000	5.6	897,460	
				3,195,080
Towers (321 ft. high):				
Structural steel	35,720,000	5.6	2,000,320	
Steel castings	3,360,000	9.0	302,400	
				2,302,720

Anchorage steel:				
Eye-bars and riveted work	2,075,000	5.0	103.750	
				103,750
Total steel work				$7,887,300
Tower foundations:				
Masonry in pier (cu. yds.)	12,890	$18	232,020	
Concrete in cribwork (cu. yds.)	25,300	$7.50	189,750	
Concrete in caisson (cu. yds.)	4 080	$12	48,960	
Timber in cribwork (M. ft. B.M)	620	$45	27,900	
Steel in caisson (lbs.)	1,150,000	4½c.	51,750	
Earth excavation (cu. yds.)	28,600	$4	114,400	
				1,329,560
Anchorages:				
Masonry (cu. yds.)	63,500	$18	1,143,000	
Concrete in foundation (cu. yds.)	5,920	$12	71,040	
				2,428,080
Total substructure				$3,757,640
Grand total				$11,644,940

SUSPENSION BRIDGE. SPAN = 2250 FT.

Estimate of Cost.

Material.	Quantity.	Price.	Amount.	Total.
Cables, etc.:	lbs.	c.	$	$
8 cables at 317 sq. in.	40,220,000	12.5	5,027,500	
Suspenders	3,656,000	10.0	365,600	
Steel castings (cable bands, etc.)	5,600,000	9.0	494,000	
				5,887,100
Suspended superstructure:				
Truss chords (nickel steel)	53,960,000	8.0	4,316,800	
Webs and bracing (struct. steel)	17,934,000	5.6	1,004,300	
Floor system (structural steel)	24,039,000	5.6	1,346,180	
				6,667,280
Towers (454 ft. high):				
Structural steel	77,960,000	5.6	4,365,760	
Steel castings	6,480,000	9.0	583,200	
				4,948,960

Anchorage Steel:				
Eye-bars and riveted work	4,945,000	5.0	247,250	247,250
Total steel work				$17,750,590
Tower foundations:			278,820	
Masonry in pier (cu. yds.)	15,490	$18	280,500	
Concrete in cribwork (cu. yds.)	37,400	$7.50	62,400	
Concrete in caisson (cu. yds.)	5,200	$12	35,550	
Timber in cribwork (M. ft. B.M.)	790	$45	104,400	
Steel in caisson (lbs.)	2,320,000	4½c.	156,000	
Earth excavation (cu. yds.)	39,000	$4		1,835,340
Anchorages:				
Masonry (cu. yds.)	109,000	$18	1,962,000	
Concrete in foundation (cu. yds.)	10,400	$12	124,800	
				4,173,600
Total substructure				$6,008,940
Grand total				$23,759,530

SUSPENSION BRIDGE—SPAN = 3000 FT.

Estimate of Cost.

Material.	Quantity.	Price.	Amount.	Total.
Cables, etc.:	lbs.	c.	$	$
12 cables at 349 sq. in.....	92,280,000	12.5	11,535,000	
Suspenders.....	9,040,000	10.0	904,000	
Steel castings (cable bands, etc.)	13,650,000	9.0	1,228,500	
				13,667,500
Suspended superstructure:				
Truss chords (nickel steel).....	104,400,000	8.0	8,352,000	
Webs and bracing (struct. steel)	29,418,000	5.6	1,647,410	
Floor system (structural steel)..	32,052,000	5.6	1,794,910	
				11,794,320
Towers (587 ft. high):				
Structural steel.....	179,248,000	5.6	10,037,890	
Steel castings.....	9,600,000	9.0	864,000	
				10,901,890

Anchorage Steel:				
Eye-bars and riveted work.....	8,180,000	5.0	409,000	
				409,000
Total steel work.....	.....	.....	.....	$36,772,710
Tower foundations:				
Masonry in pier (cu. yds).....	26,300	$18	473,400	
Concrete in cribwork (cu. yds.)..	67,800	$ 7.50	508,500	
Concrete in caisson (cu. yds.)...	8,230	$12	98,760	
Timber in cribwork (M. ft. B.M.)	1,080	$45	48,600	
Steel in caisson (lbs.).....	2,650,000	4½c.	119,250	
Earth excavation (cu. yds.).....	65,000	$ 4	260,000	
				3,017,020
Anchorages:				
Masonry (cu. yds.).....	183,200	$18	3,297,600	
Concrete in foundation (cu. yds.)	16,650	$12	199,800	
				6,994,800
Total substructure.....	.....	.....	.....	$10,011,820
Grand total.....	.....	.....	.....	$46,784,530

CHAPTER IV

CONCLUSIONS FOR SUSPENSION BRIDGES

ART. 15

EMPIRIC FORMULÆ FOR WEIGHTS OF SUSPENSION BRIDGES

FROM the results of the preceding designs, we may construct semi-empirical formulæ for the weights of the different parts of a suspension bridge. Such formulæ will be useful in preparing estimates for other spans and loadings; also in drawing general conclusions as to maximum and limiting spans.

Let the weights (per linear foot) of live-load, cable, suspenders, truss (including bracing) and floor be represented by L, C, S, T and F, respectively. The weight of the cable is evidently proportional to the total load it sustains and to the length of span, provided the

rise-ratio remains unchanged. The weight of the suspenders is likewise proportional to the load they carry and to the span, as a longer span requires a proportionally greater average length of suspenders. The weight of the truss may be considered in two parts: the larger portion provides for the live-load and is proportional to that load and to the span length, if the depth ratio remains constant; the other part of the truss-weight represents the contribution of the wind-stresses and will vary with the square of the span if the width of bridge remains constant. The weight of the floor system is evidently proportional to the assumed live-load, and is practically independent of the length of span. Introducing the necessary coefficients of proportionality, the above relations may be formulated as follows:

$$\left.\begin{aligned} C &= a(C+L+S+T+F)\cdot l \\ S &= b(L+S+T+F)\cdot l \\ T &= c(L)\cdot l + d\cdot l^2 \\ F &= e\cdot L \end{aligned}\right\} \quad . \quad (1)$$

Substituting the values of weights and spans from the preceding designs, the spans being measured in units of 1000 ft., the above relations yield the following values for the undetermined coefficients.

	S. B. No. 1.	S. B. No. 2.	S. B. No. 3.	Mean.
$a=$	.070	.066	.068	.068
$b=$	.0077	.0073	.0074	.0075
$c=$	.35	.34	.36	.35
$d=$	105	105	105	105
$e=$	.83	.84	.85	.84

The uniformity in the above coefficients for the different spans confirms the rationality of the relations assumed above. Introducing the empirical constants in those relations, we obtain the following formulæ for suspension bridges:

$$\left.\begin{aligned} C &= .068(C+L+S+T+F)\cdot l \\ S &= .0075(L+S+T+F)\cdot l \\ T &= .35\cdot L\cdot l + 105\, l^2 \\ F &= .84\, L \end{aligned}\right\}. \quad (2)$$

Art. 16

Maximum Span for Cable

The theoretical limiting span for a suspension bridge is the span at which the cable-section becomes infinitely large in proportion to the load it can carry. The defining equation for this condition is, therefore,

$$\frac{A_0}{g_1} = \infty \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where g_1 is the intensity of the total load suspended from the cable.

If g_0 = the weight of the cable per linear horizontal unit, and n = the rise-ratio $= \frac{f}{l}$, the maximum tension in the cable will be

$$T = H \cdot \sec \alpha = \left(g_0 + g_1\right)\frac{l^2}{8f}\sqrt{1+16n^2}. \quad (2)$$

The weight g_0 is given by the equation

$$g_0 = \left(1 + \frac{8}{3}n^2\right)\frac{T}{s_0} \cdot \gamma_0 \quad . \quad . \quad (3)$$

where s_0 is the unit working stress and γ_0 is the weight per linear foot per sq.in. of cross-section of the cable. From Eqs. (2) and (3) we obtain

$$A_0=\frac{T}{s_0}=\frac{l\cdot\sqrt{16+\frac{1}{n^2}}}{8s_0-\left(1+\frac{8}{3}n^2\right)\cdot\gamma_0\cdot l\cdot\sqrt{16+\frac{1}{n^2}}}\cdot g_1 \tag{4}$$

Introducing the condition for maximum span, as defined by Eq. (1), we obtain the relation

$$8s_0-\left(1+\frac{8}{3}n^2\right)\cdot\gamma_0\cdot\sqrt{16+\frac{1}{n^2}}\cdot l=0.$$

Hence,

$$l_{max}=\frac{8s_0}{\left(1+\frac{8}{3}n^2\right)\sqrt{16+\frac{1}{n^2}}\cdot\gamma_0}. \tag{5}$$

This equation shows that the maximum limiting span is independent of the live-load or weight of the stiffening truss. Equating the first derivative of the second member of (5) to zero, we find

the rise-ratio giving the absolute maximum span to be

$$n=\frac{f}{l}=.306. \quad . \quad . \quad . \quad (6)$$

With this value, Eq. (5) becomes

$$l_{max}=1.085\frac{s_0}{\gamma_0}. \quad . \quad . \quad (5')$$

Numerical Values. For very heavy cables, the weight of the cable-wrapping may be neglected; hence γ_0 is simply the density-factor of steel $=3.4$.

The extreme limit which no cable-span can ever exceed is that for which s_0 equals the elastic limit of the best cable material or $s_0=180{,}000$. Eq. (5′) then gives

$$\text{Extreme } l_{max}=65{,}520 \text{ ft.} \quad . \quad (7)$$

For practical purposes it is necessary to introduce a safety factor in the working stress; reducing the latter to $s_0=60{,}000$, we obtain, by Eq. (5′),

$$l_{max}=21{,}840 \text{ ft} \quad . \quad . \quad . \quad (7')$$

This span with a rise-ratio of $n=.306$, would require towers about 7000 ft.

high; the above result, therefore, is of no practical significance.

Replacing the above value of n by the economic ratio

$$n=\frac{f}{l}=\frac{1}{8}, \quad . \quad . \quad . \quad . \quad (6')$$

and retaining $s_0=60{,}000$, Eq. (5) yields

$$\text{Practical } l_{\max}=15{,}160 \text{ ft.} \quad . \quad (7'')$$

This value gives the limit of span which may be approached, but not exceeded, by successively reducing the amount of extraneous load for any given cable-section or by increasing the cable-section for any given load so that $\frac{A_0}{g_1}=\infty$. At the maximum span given by Eqs. (7), (7') or (7''), the cable-section may have any finite value so long as there is no load suspended from it. The addition of the smallest load will necessitate reducing the span or else increasing the section A_0 to infinity.

It may be noted that the limiting span increases directly with the working

stress s_0 and inversely with the cable-weight factor γ_0.

In comparison with the above limiting values, it is of interest to consider the largest existing cable-span. This is a cableway at Caperton, W. Va., built in 1898, and having a span of $l=2100$ ft., or only about one-seventh of the span-length which may yet be attained.

Art. 17

Maximum Span for Suspension Bridges

From Eq. (2) of Art. 15, we obtain the following empirical expression for the weight of the cable (C) in terms of the suspended loading and the length of span (l):

$$C=\frac{.068l}{1-.068l}(L+S+T+F). \quad . \quad (1)$$

Substituting the empirical expressions for S, T and F in terms of the live-load L, Eq. (1) becomes, practically,

$$C=\frac{.068l(1.84L+.35Ll+105l^2)}{(1-.068l)(1-.0075l)}. \quad (2)$$

The theoretical maximum span for the suspension bridge is the span at which the cable-section becomes infinite. This condition is realized when the denominator of Eq. (2) reduces to zero, or

$$l_{max} = \frac{1}{.068} = 14{,}700 \text{ ft.}$$

This is the upper limit of feasible spans for suspension bridges and represents the span at which the suspension bridge ceases to be self-supporting. It is somewhat smaller than the maximum span-limit (15,160 ft.) for a simple cable, as determined in the preceding article, simply because the weight of cable wrapping and fastenings was not considered in that investigation.

Another method of deducing the limiting span, possessing the advantage of greater generality inasmuch as it is applicable to other forms of bridge-structures, is as follows: Let C_1 = the weight of the cable and W_1 = the total suspended load, per linear foot, for any span (l_1). For any other span

(l), the weight of the cable is given by the proportion

$$\frac{C}{C_1}=\frac{(C+W)l}{(C_1+W_1)l_1}. \quad . \quad . \quad (3)$$

Solving this equation, we obtain

$$C=\frac{WC_1l}{(C_1+W_1)l_1-C_1l}. \quad . \quad (4)$$

For $C=\infty$, the denominator of the above expression must reduce to zero, or

$$l_{max}=\frac{(C_1+W_1)\cdot l_1}{C_1}. \quad . \quad . \quad (5)$$

This formula enables the limiting span to be calculated from the results of any single design.[1] In the designs at hand,

$l=$	1,500	2,250	3,000
$C=$	1,286	2,160	3,720
$W=$	11,162	12,522	14,418
$\therefore l_{max}=$	14,300	15,300	14,600 ft.

[1] The same formula, with C denoting the weight of the arch-rib, was used by the writer in determining the maximum spans for steel and concrete arches. See Thesis submitted for the Degree of C.E. at Columbia University.

The practical agreement between these values indicates the reliability of the above method of determining the maximum span from a single design. Taking the mean of the above values, there results

$$l_{max} = 14{,}700 \text{ ft.}$$

a value identical with that yielded by the first method of this article.

Although the above is the limiting feasible span, defined by $C = \infty$, it is evident that the sections will become too large for practical construction long before that length of span is reached. In order to determine the *practical* maximum span, a value for the maximum cable-section must first be fixed.

The cables of the three largest suspension spans, viz., the Brooklyn, Williamsburg and Manhattan Bridges, are $15\frac{5}{8}$, $18\frac{3}{4}$ and $21\frac{1}{2}$ inches in diameter, respectively. Cables with a diameter of 24 inches were used in one of the designs in this investigation, but it is doubtful whether any larger diameter

can be put together without excessive difficulty of manipulation and uncertainty of proper distribution of stress among the different strands.

With the exception of some old chain bridges, there is no bridge on record with more than six cables. Twice that number, or four groups of three, were used in one of the writer's designs, but it is extremely improbable that a greater number than 16 could be practically combined in one structure.

Assuming, then, a section of 16 cables of 24 inches diameter for the bridge of maximum span, we have

$$C_{\max} = 20{,}000 \text{ lbs.}$$

Solving Eq. (2), Art. 15, for the length of span in terms of the cable-weight and live-load, there results

$$l = \frac{C}{.0734C + .1253L + .0238Ll + 7.14l^2}. \quad (6)$$

Substituting in this equation the maximum value of C, as just established, and assuming different values of the live-

load, we obtain the following values for the maximum span:
For

$L=0$, $l_{max}=9{,}500$ ft.
$L=10{,}000$, $l_{max}=4{,}900$ ft.
$L=15{,}000$, $l_{max}=4{,}000$ ft.
$L=20{,}000$, $l_{max}=3{,}500$ ft.

The first of the above results simply signifies that 9500 ft. is the span at which the cable will just be able to support the wind-bracing. As this condition is one of zero loading, it may be omitted from practical consideration. Furthermore, it is hardly probable that a structure of the magnitude under investigation would ever be planned for a lighter loading than about 10,000 lbs. per linear foot, particularly when it is remembered that this would require about 10 lbs. of steel for every pound of useful load.

It will therefore suffice to restrict the conditions of the problem to the practical limits of $L=10{,}000$ to $L=20{,}000$ lbs. of live-load per linear foot. We thus find that *the maximum practical*

span for suspension bridges ranges from 3500 *to* 4900 *ft., depending upon the assumed live-load.*

Art. 18

Empiric Formula for Cost of Suspension Bridges

The expression for the cost of any span (l) will be assumed of the general form

$$C = al + bl^2 + cl^3. \quad . \quad . \quad . \quad (1)$$

Since we have but three values determined for C by actual design, the formula is limited to an equal number of terms.

From the results of the preceding estimates, we have

$C = \$11{,}645{,}000$ for $l = 1500$ ft.
$C = \$23{,}760{,}000$ for $l = 2250$ ft.
$C = \$46{,}785{,}000$ for $l = 3000$ ft.

Substituting these values in (1), and solving the resulting three equations

for the unknown coefficients, we obtain

$$C = 8900l - 3.77l^2 + .0020l^3. \quad . \quad (2)$$

as the general cost-formula for suspension bridges. This gives the combined cost of steel work and substructure for any span for an assumed live-load of 18,000 lbs. per linear foot. For any other loading, the above coefficients should be changed in proportion.

Art. 19

Economic Span for Suspension Bridges

As established in the preceding article, the cost of a suspension bridge is given by the expression

$$C = 8900l - 3.77l^2 + .0020l^3. \quad . \quad (1)$$

Of this cost, about 65 per cent represents the steel work and the remainder provides for the masonry and anchorages.

A study of the contracts for recent long-span bridges shows an additional cost of about 20 per cent for pavements,

tracks, railings, ornamental work, electric lighting, etc. To this must be added the cost of terminal structures and real estate for the approaches. In a city structure the last item, as in the case of the Manhattan Bridge, may amount to more than the cost of the bridge itself. A fair average value for this item is about 100 per cent. Adding the above items, we find

$$\text{Total first cost of bridge} = 220\%\ C. \quad (2)$$

The rate of interest will be assumed at 5 per cent. The cost of maintenance, including repairs, painting, lighting, etc., averages about $4\frac{1}{2}$ per cent of the cost of the superstructure. In addition, a certain annuity must be set aside for the periodic renewal of the superstructure. The foundations may be permanent, but the steel work has a limited life. The life-periods of various suspension bridges, or the periods before reconstruction or removal, whether terminated by failure or increased traffic demands, have been as follows:

1827	[1] Hammersmith (London)	.55	years.
1829	[2] Regnitz (Bamberg)	.59	"
1834	[2] Freiburg (Switzerland)	18	"
		29	"
1839	[1] Weser (Hamlin)	.51	"
1845	[1] Neckar (Mannheim)	.46	"
1850	Fairmount (W. Va.)	.40	"
1851	[3] Niagara (Old S. B. at Lewiston)	.13	"
1852	[4] Charleston (W. Va.)	.52	"
1855	[3] Niagara Falls (Railway Bridge)	.42	"
1867	[5] Cincinnati (Ohio R.)	.31	"
1868	[1] Moldau (Prague)	.32	"
1869	[3] Niagara Falls (Hwy. Bridge)	19	"
		1	"
		17	"
1877	[6] Point Bridge (Pittsburg)	.28	"
	Mean Life Period	.31	years.

[1] Melan, "Konstruktion der Hängebrücken" (Leipzig, 1906), p. 204.

[2] Nouv. ann. de la constr., 1881. Also Riese, "Die Ingenieurbauwerke der Schweiz."

[3] *Eng. Record*, 1897 (Apr. 24); 1899 (Aug. 26).

[4] *Eng. News*, 1905 (Feb. 2). *Eng. Record*, 1904 (Dec. 24).

[5] *Eng. Record*, 1898 (Sept. 10, Nov. 26).

[6] *Eng. Record*, 1905 (May 6–13).

This value, or 30 years in round numbers, will be adopted as the probable life of the steel work of a suspension bridge. It is true that the newer bridges possess the advantages of improved material and construction, but the more severe traffic to which they are subjected and the smaller margin of safety provided in their design prevent them from attaining as long a life as some of the old structures. For a railroad bridge strained to its full capacity, the above value of the life-period is certainly not too small. At 5 per cent compound interest, a sinking fund to meet the cost of renewal in 30 years will require an annuity of 1.505 per cent. (See Annuity Tables.) The annual charge against the bridge will therefore consist of the following items:

Interest charge	$= 5\% \times 220\%\ C = 11\%\ C$
Repairs and maintenance	$= 4.5\% \times 65\%\ C = 3\%\ C$
Depreciation = Annuity for renewal in 30 years $= 1.505\% \times 65\%\ C$	$= 1\%\ C$
Total annual charge	$= 15\%\ C$

The limiting economic span is that at which the revenue from traffic (T) just balances the annual cost of the structure. We may therefore write

$$15\%C = T. \quad . \quad . \quad . \quad (3)$$

as the defining condition for the economic span.

In determining the maximum traffic returns (T) to be expected from a long-span bridge, we are guided by the following considerations: The Brooklyn Bridge now carries about 118,000,000 paid passengers per annum.[1] The maximum daily is 30 per cent greater than this rate and the maximum hourly is 500 per cent greater still.[2] The Williamsburg Bridge, opened 20 years later, is a close competitor, with a passenger traffic of 75,000,000 per annum[3]; at the present rate of increase, it will

[1] Report of Public Service Comm., 1st District, N. Y., 1909.

[2] *Engineering Record*, 1910 (June 11).

[3] *Engineering News*, 1910 (Jan. 27).

soon equal the older structure in volume of traffic. The usefulness of both these structures is steadily growing, despite the opening of several competing routes of communication across the same river. Each bridge helps to build up the districts which it connects, thereby creating increased traffic for itself. Thus there were 9,000,000 passengers crossing the Brooklyn Bridge in 1884,[1] 42,000,000 passengers in 1893,[1] and 118,000,000 passengers in 1909.[2] The total number of passengers annually crossing the East River increased 500 per cent in the 17 years after the opening of the Brooklyn Bridge,[3] and 240 per cent more since the Williamsburg Bridge was opened.[4] These facts indicate that

[1] Statement of Chas. Macdonald to Bd. of Eng. Officers, 1894; also *Engineering News*, 1893 (Feb. 23).

[2] Report of Public Service Comm., 1st Distaict, N. Y., 1909.

[3] Lindenthal in Discussions at the N. Y. R. R. Club, Apr. 18, 1901.

[4] *Engineering News*, 1910 (Jan. 27).

any large bridge, if judiciously located, will ultimately get all the traffic it can accommodate.

In the absence of any better guide, let us take the amount of travel on the Brooklyn Bridge as the maximum traffic to be expected on any other long-span structure. This value may justly be augmented by 50 per cent for the greater capacity of a six-track bridge, but this additional profit will be disregarded to compensate for the early years of undeveloped traffic. We will therefore assume 118,000,000 passengers using the bridge in a year. For a span of the length under consideration, a toll of 5 cents per passenger would be an equitable rate. The same charge is now made for ferry or tunnel transportation across the Hudson River, and bridge travel would surely be preferred for its greater speed and comfort.

At night, when the passenger traffic is a minimum, the tracks can be used for the transportation of freight. There are about 6000 cars of freight, inbound

and outbound, at Jersey City daily, and about half of this belongs to New York. We may therefore safely count on at least 1500 cars of freight daily crossing a six-track bridge over the Hudson River or any similar location.

The total annual traffic over a long-span bridge may therefore attain the following value:

118,000,000 passengers at 5 cents	= $5,900,000
547,500 cars of freight at $4	= 2,190,000
$\therefore T$ = total annual revenue	= $8,090,000

Substituting this value in eq. (3), we obtain

$$C = \$54{,}000{,}000$$

as the maximum economic cost for a long-span bridge. With this value of C, the solution of eq. (1) yields

$$\text{Economic } l = 3170 \text{ ft.}$$

In the case of the proposed Hudson River Bridge, of somewhat shorter span, calculations indicated a profit of less

than 1 per cent on the investment,[1] thus confirming the above result.

Conclusion. The limiting economic span for suspension bridges is about 3170 *ft., and will be less wherever the probable traffic returns are smaller than assumed in this investigation.*

[1] H. G. Prout in Discussions at the N. Y. Railroad Club, Apr. 18, 1901.

CHAPTER V

STUDY OF CANTILEVERS

ART. 20

CANTILEVER BRIDGES—HISTORICAL SKETCH

IN adaptability to long spans and possibility of erection without falsework, the cantilever is the sole competitor of the suspension bridge. Both of these types have attained prominence by remarkable examples of design and construction. With but one exception, however, the suspension type has never been employed for fast railway traffic. Its use has been confined to highway bridges or wherever æsthetic requirements prevailed. The banner bridges of the suspension type, those over the East River, have been attributed to "an ingrained fad of the New York

Department of Bridges." For long railway spans, the cantilever has almost invariably been given the preference on account of its superior rigidity at a given cost. The longest span in the world (Forth Bridge 1710 ft.) is of the cantilever form, and the Quebec Bridge, now under construction, will raise the record for length of span to 1800 ft.

Although cantilever design is a comparatively recent development in engineering, the idea is by no means a new one. Bridges of logs, put together on the cantilever principle, have been used in tropical countries since prehistoric times.[1] In 1783, a wooden cantilever bridge of 112-ft. span was reported by travelers in Thibet.[2] An 1800-ft. "flying pendant lever bridge" to cross the East River and a 3000-ft. span across the North River were proposed by Pope in 1810.[3] Fairbairn's proposal

[1] *Van Nostrand's Magazine*, Jan., 1886.

[2] Pope, Treatise on Bridge Architecture, New York, 1811.

[3] Ibid.

for the Britannia Bridge, in 1845, was a cantilever design.[1] Stephenson, in 1846, and Edwin Clark, in 1850, suggested the cantilever idea.[2] In the latter year Sir John Fowler built a wooden model to illustrate the form of construction.[3] In 1859 Prof. Ritter of Hanover proposed cutting the chord of a continuous truss at the points of contraflexure and worked out the stresses in the resulting cantilever structure for a span of 526 ft. The first cantilever design actually constructed, however, was a bridge of 124 ft. span over the Main River near Hassfurt, designed and built by Gerber in 1867.[4] On this account, cantilevers are known as "Gerber Bridges" on the continent. In 1871, Fowler and Baker built two cantilever spans of 800 ft. over the Severn, and in 1873 Baker designed a

[1] *Engineering* (London), Mar. 5, 1886.

[2] *Engineering* (London), Feb. 28, 1890.

[3] Proc. of the Inst. of C. E., IX, p. 256.

[4] Mehrtens, "Der deutsche Brückenbau im 19. Jahrundert," Berlin, 1900.

cantilever ferry bridge of 650 ft. span over the Tees.[1] The first cantilever bridge for railway traffic, a span of 148 ft., was built in 1876 over the Warthe at Posen.[2] In the same year, the first American cantilever, the Kentucky Viaduct, was built by C. Shaler Smith.[3] In this structure and in the Niagara Cantilever built in 1883 by C. C. Schneider for the Michigan Central Railroad, the possibility of erection without falsework was first demonstrated.[4] In 1881, four years after the completion of the Kentucky Viaduct, the final designs for the Forth Bridge were approved. The successful completion of that remarkable structure in 1889[5] marked the end of the experimental period for the cantilever and served to fix that type in its present dominant position in long-span construction.

[1] *Engineering News*, Nov. 24, 1904.
[2] *Engineering* (London), Feb. 28, 1890.
[3] Trans. Am. Soc. of C. E., Nov., 1878.
[4] Trans. Am. Soc. of C. E., Nov. 1885.
[5] *Engineering* (London), Dec. 6, 1889.

A table of the most noted cantilever bridges with their principal dimensions is appended. It will be observed that only three cantilevers exceed 1000 ft. in span, whereas eight suspension bridges have exceeded this value. It is mainly between the limits of 500 and 1000 ft. that cantilevers predominate. The high values of the ratio of dead-load to live-load are significant, indicating a greater expenditure of metal than is required in suspension bridges.

It will also be noticed that there are comparatively few cantilevers below 500 ft. span, that being the domain of the ordinary truss. With increasing span-length, however, the cantilever bridge becomes superior to the simple truss because of the increasing significance of the following advantages.

1. No obstruction of the channel during erection and the saving of the cost of falsework.

2. Lower economic depth of truss.

3. Smaller required width resulting in a saving in the piers and in the floor system.

TABLE II—NOTABLE

Date.	Name.	Location.	Engineer.
1908	[1] Quebec.....	St. Lawrence R.	Cooper.....
1889	Forth.....	Scotland.....	Fowler & Baker
1908	Queensboro..	East R., N. Y...	Dept. of Br.
....	Lansdowne..	India.....	.....
1903	Monongahela.	Pittsburg.....	Boller & Hodge
1892	Memphis	Mississippi R. ..	Morison.....
1910	Beaver.....	Ohio R., Pa....	.....
1903	Mingo.....	Ohio R.....	Boller & Hodge
1905	Thebes.....	Miss. R., Ill. ...	Modjeski & Noble.....
1904	Ruhrort.....	Rhine R., Germany.....	.....
1891	Red Rock....	Colorado R.....	.....
1902	Marietta.....	Ohio R., Ohio. .	.....
1902	Czernavod...	Danube R.....	.....
1887	[2] Poughkeepsie	Hudson R.....	.....
1906	Long Lake ..	New York	.....
1903	Connel Ferry.	Scotland.....	Barry.....
1897	Francis Joseph	Budapest.....	.....
1883	Niagara.	Niagara Falls...	Schneider.....
1876	[2] Kentucky R.	Ohio.....	C. S. Smith....
1883	Fort Snelling	Mississippi R., Minn.....	Schneider.....

[1] Collapsed 1908, before completion.
[2] Reconstructed 1903.

4. More favorable distribution of the dead-load, the material being massed toward the piers.

CANTILEVER BRIDGES

Sp'n $=L$	S.S. $=l$	C.A. $=m$	A.A. $=n$	Width	Depth.	Loading.		
						L.L.	D.L.	D.L./L.L.
1800	675	562½	500	67	98–315	13000	26000	2.0
1710	350	680	680	32–120	50–350	4480	21000	4.7
1182	0	591	630	60	45–185	8440	27000	3.2
820	200	310	248					
812	360	226	346	32	60–126	9000	9000	1.0
790	452	169	621	30	78	4000	7000	1.75
769	285	242	320	.....	.....	.....	11000	
700	310	195	298	.....	.....	9000	8000	0.9
671	366	152½	521	32	50–75	8000	10000	1.25
667	443	112	390	36	46–82			
660	....	.....	.....	25				
650	270	300	600	27				
623								
548	208	170	525[1]	30	38–75	6000		
525	175	175	0	24	20–60	920	840	0.9
524	232	146	106	21	30–118	4000	7620	1.9
514	102	206	257			6000		
495	120	187½	207½	28	.....	3000		
375	300	75	375	18	38			
							2530	0.85
315	105	105	105					

[1] Replaced by truss bridge, 1910.

5. More favorable distribution of the wind-load, for the same reason.

6. Decreased wind-load stresses.

In order to establish data for determining the economic and maximum spans, complete designs and estimates will be made for cantilevers of three different spans: 1000, 1500 and 2000 ft., respectively. The condition of loading and specifications for allowable working stresses will be assumed the same as for the suspension bridges previously designed.

Before we can proceed with the design of the cantilevers, it is necessary to find the most favorable solutions of the following problems.

1. Economic span-ratios for cantilevers.
2. Economic width for cantilevers.

Art. 21

Economic Span-ratios for Cantilevers

Let l = length of suspended span;
m = length of cantilever arm;
n = length of anchor arm.

Then the total channel span will be

$$L = l + 2m \quad . \quad . \quad . \quad . \quad (1)$$

and the total length of structure will be

$$S = l + 2m + 2n \quad . \quad . \quad . \quad (2)$$

Assuming that the weight per linear foot of truss in any span or arm is equal to some constant factor times the length of that span or arm, let

a = suspended span factor;
b = cantilever arm factor;
c = anchor arm factor.

Then the weight of the whole structure will be

$$W = al^2 + 2bm^2 + 2cn^2 \quad . \quad . \quad (3)$$

This will be a minimum for

$$al = bm = cn \quad . \quad . \quad . \quad (4)$$

Hence the economic lengths of the suspended span, cantilever arm and anchor arm are inversely proportional to the

respective dead-load factors; in other words, *the economic lengths are such as to make the weight per linear foot uniform over all parts of the bridge.*

From (1), (2) and (4) we obtain [1]

$$l=\frac{b}{b+2a}\cdot L, \quad m=\frac{a}{b+2a}\cdot L,$$

$$n=\frac{ab}{c(b+2a)}\cdot L. \quad (5)$$

Numerical Values. The weight factors a, b and c should be obtained by actual computation of weights of structures similar to those under consideration. The resulting values will, of course, be found to vary more or less with different loadings and span-ratios. By omitting from consideration all spans of unusual proportions, and reducing the results to a common assumed loading (=18,000 lbs. per linear foot as in the following designs), the writer obtained

[1] Cf. Burr's solution of the problem in his "Stresses in Bridges and Roof Trusses, etc." (Wiley, 1908), App. V., p. 472.

the following average values of the truss-weight factors:

$$a=14,\ b=42,\ c=21.^{1}$$

Substituting these values in Eqs. (5), we obtain

$$l=0.60\ L,\ m=0.20\ L,\ n=0.40\ L,$$

for the economic span-ratios of a cantilever bridge. Prof. Burr [2] recommends values of

$$l=0.5 \text{ to } 0.55\ L;\quad m=0.20\ L;$$
$$n=0.42 \text{ to } 0.5\ L.$$

An exact theoretical solution of the problem,[3] eliminating the use of empirical constants, shows that the total moment areas will be a minimum for

$$l=0.68\ L;\quad m=0.16\ L,\quad n=0.37\ L.$$

[1] Compare these values with those yielded by the writer's designs, viz.: $a=13.6$, $b=42.3$, $c=21.7$.

[2] Burr's "Stresses in Bridges and Roof Trusses, etc." (Wiley, 1908), App. V, p. 472.

[3] Marburg in Proc. of Eng. Club of Phila., July, 1896.

This solution is practically defective, however, in disregarding the web-members and lateral bracing and in assuming uniform dead-load over the whole structure.

A study of the spans of the cantilevers tabulated in a preceding article, omitting those of extraordinary form, yields the following results:

Extreme values:

l/L	m/L	n/L
.20 – .66	.17 – .40	.28 – .78

Mean values:

.38	.31	.45

These values compared with those established above indicate that, in past practice, the suspended span as a rule has been made too long and the cantilever arms too short for the best economy.

For the designs in this investigation, the writer has adopted a compromise between the dictates of theory and those of conformity with past practice. The

lengths of the suspended spans will be made for

$$\begin{array}{rrrr} L = & 1000 & 1500 & 2000, \\ l = & 500 & 650 & 800, \end{array}$$

so that

$$l/L = \quad 0.5 \qquad 0.43 \qquad 0.4.$$

The suspended span is thus made a diminishing fractional part of the total span as the latter increases in length; otherwise the length of the suspended truss would become prohibitive before the limiting cantilever span is reached. (See Art. 26.)

The lengths of the anchor arms will be made

$$n = 0.4L$$

thereby conforming with the ratio indicated by both theory and practice. This determination of the best length for the anchor arm, however, is purely academic as, in any actual design of a cantilever, the location of the piers is

determined by natural conditions and the requirements of navigation rather than from any theoretical investigations.

Art. 22

Minimum Width for Cantilevers [1]

On account of the saving in piers and floor members, it is desirable that the width of any bridge structure should be a minimum consistent with the demands of lateral rigidity. In simple truss bridges it is generally considered that adequate stiffness is provided by a distance c. to c. of trusses of $\frac{1}{18}$ of the span. The minimum width for cantilevers may then be defined as that width which will insure the same degree of rigidity as the above.

Using the same notation as in the preceding article, the total length of span for a cantilever bridge is denoted by

$$L=l+2m \quad . \quad . \quad . \quad .(1)$$

[1] Adapted from an analysis given to his classes by Prof. Burr.

If a lateral load, p per linear foot, acts on the structures, the central deflection will be

$$h=\frac{p}{EI}\left[\frac{5}{384}l^4+\frac{l}{2}\frac{m^3}{3}+\frac{m^4}{8}\right] \quad . . \quad (2)$$

This reduces to

$$\left.\begin{aligned} h&=.00260\frac{p}{EI}L^4 \text{ for } l=0.5L \\ \text{and} \\ h&=.00315\frac{p}{EI}L^4 \text{ for } l=0.4L \end{aligned}\right\} . \quad (3)$$

In a simple truss of span l' the central deflection is given by

$$h=\frac{5}{384}\cdot\frac{p}{EI}l'^4. \quad . \quad . \quad . \quad . \quad (4)$$

With equal rigidity, the simple truss and the cantilever must suffer the same deflections. Equating the right-hand members of Eqs. (3) and (4), there result

$$\left.\begin{aligned} l'&=.668L \quad \text{for} \quad l=0.5L \\ \text{and} \\ l'&=.701L \quad \text{for} \quad l=0.4L \end{aligned}\right\} . \quad (5)$$

The distance c. to c. of trusses should not be less than

$$w = \frac{l'}{18},$$

or, substituting the relations of Eq. (5)

$$\text{Minimum } w = \frac{1}{27} \text{ to } \frac{1}{25}L. \quad . \quad (6)$$

It is interesting to note here that the mean width-ratios for the structures listed in the Table of Cantilevers is

$$w = \frac{1}{23}L,$$

a value on the safe side of the required ratio.

In the designs at hand a ratio of $w = \frac{1}{25}L$ will be adopted except in the shortest span where the required space for the roadway will necessitate a slightly greater width-ratio, namely,

$$w = \frac{1}{20}L.$$

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CHAPTER VI

DESIGN OF CANTILEVERS

ART. 23

PRINCIPAL DATA FOR DESIGN OF CANTILEVER BRIDGES

	No. 1.	No. 2.	No. 3.
L = total span = $m+l+m$ =	ft. 1000	ft. 1500	ft. 2000
l = suspended span =	500	650	800
m = cantilever arm =	250	425	600
n = anchor arm =	400	600	800
w = width c. to c. of trusses =	50	60	80
d = depth of suspended truss =	50–85	75–110	100–135
h = depth at towers =	150	225	300

Load: $L.L = 18{,}000$ lbs. p.l.f.
Wind = 30 lbs. per sq.ft.

Working Stresses.	*Tension.*	*Compression.*
Structural steel	20,000	$\dfrac{20{,}000}{1+\dfrac{l^2}{8000r^2}}$
Nickel steel	30,000	$\dfrac{30{,}000}{1+\dfrac{l^2}{8000r^2}}$

For erection stresses, the above may be increased by 20 per cent.

Art. 24

Estimate of Cost

As there are no points of special interest involved in the design of the cantilever bridges, the details of the computations and the tables of stresses are here omitted. The total quantities of material in the different parts of the structures are presented in the following estimates of cost. In these estimates, the same unit prices are used as in the case of the suspension bridges. (See Art. 14.)

CANTILEVER BRIDGE No. 1.

($L = 1000$.)

ESTIMATE OF COST.

Material.	Quantity. lbs.	Price. cts.	Amount. $
Suspended Span.			
Truss, structural steel	1,446,400	5.6	80,998
nickel steel	3,074,800	8.0	245,984
Floor, structural steel	2,837,500	5.6	158,900
Bracing, struct. steel	1,132,800	5.6	63,437
Pins, nickel steel	246,000	10.0	24,600
Total			573,919
Cantilever Arm.			
Truss, structural steel	2,051,000	5.6	114,856
nickel steel	2,390,600	8.0	191,248
Floor, structural steel	1,426,000	5.6	79,856
Bracing, struct. steel	593,800	5.6	33,253
Pins, nickel steel	179,200	10.0	17,920
Total			874,266
Anchor Arm.			
Truss, structural steel	2,559,400	5.6	143,326
nickel steel	3,647 600	8.0	291,808
Floor, structural steel	2,280,000	5.6	127,680
Bracing, struct. steel	926,000	5.6	51,856
Pins, nickel steel	223,500	10.0	22,350
Total			1,274,040
Tower.			
Structural steel	5,472,000	5.6	306,432
Steel castings	514,500	9.0	46,296
Total			705,456

CANTILEVER BRIDGE No. 1.

($L = 1000$.)

ESTIMATE OF COST—(*Continued*).

Material.	Quantity. lbs.	Price. cts.	Amount. $
Anchorage.			
Eye-bars and riveted work	785,000	5.0	39,250
Total			78,500
Total steel work			$3,506,181
Main Piers.			
Pier masonry (cu.yds.)	7,590	$18	136,620
Concrete filling in cribwork	15,700	7½	117,750
Concrete filling in caisson	2,850	12	34 200
Timber in cribwork (B.M.)	404 M	45	18,180
Steel in caisson and shafts	1,270,500	4.5	57,172
Earth excavation, (cu.yds.)	18,540	$4	74,160
Total			876,164
Anchorages.			
Masonry (cu.yds.)	12,370	$18	222,660
Concrete in foundation	3,285	12	39,420
Total			524,160
Total substructure			$1,400,324
Total cost of bridge			$4,906,505

CANTILEVER BRIDGE NO. 2.

(L=1500.)

ESTIMATE OF COST.

Material.	Quantity. lbs.	Price. cts.	Amount. $
Suspended Span.			
Truss, structural steel	2,675,200	5.6	149,811
nickel steel	5,731,800	8.0	458,544
Floor, structural steel	4,387,500	5.6	245,700
Bracing, struct. steel	2,264,000	5.6	126,784
Pins, nickel steel	492,000	10.0	49,200
Total			1,030,039
Cantilever Arm.			
Truss structural steel	5,365,400	5.6	300,462
nickel steel	6,765,000	8.0	541,200
Floor, structural steel	2,960,000	5.6	165,760
Bracing, struct. steel	1,428,600	5.6	80,002
Pins, nickel steel	432,300	10.0	43,230
Total			2,261,308
Anchor Arm.			
Truss, structural steel	5,455,400	5.6	305,502
nickel steel	8,631,600	8.0	690,528
Floor, structural steel	4,180,000	5.6	234,080
Bracing, struct. steel	1,852,200	5.6	103,723
Pins, nickel steel	447,000	10.0	44,700
Total			2,757,066
Tower.			
Structural steel	16,155,000	5.6	904,680
Steel castings	1,324,000	9.0	119,160
Total			2,047,680

CANTILEVER BRIDGE NO. 2.

($L = 1500$.)

ESTIMATE OF COST—(*Continued*).

Material.	Quantity. lbs.	Price. cts.	Amount. $
Anchorage.			
Eye-bars and riveted Work	1,794,100	5.0	89,705
Total			179,410
Total steel work			$8,275,503
Main Piers.			
Pier masonry (cu. yds.)	10,810	$18	194,580
Concrete filling in crib-work	24,560	7½	184,200
Concrete filling in caisson	3,780	12	45,360
Timber in cribwork (B.M.)	558M	45	45,110
Steel in caisson and shafts	1,682,000	4.5	75,690
Earth excavation (cu. yds.)	26,480	4	105,920
Total			1,261,720
Anchorages.			
Masonry (cu. yds.)	24,380	18	438,840
Concrete in foundations	6,450	12	77,400
Total			1,032,480
Total substructure			$2,294,200
Total cost of bridge			$10,569,703

CANTILEVER BRIDGE NO. 3.

($L = 2000$.)

ESTIMATE OF COST.

Material.	Quantity. lbs.	Price. cts.	Amount. $
Suspended Span.			
Truss, structural steel.	5,142,800	5.6	287,987
nickel steel	11,070,800	8.0	885,664
Floor, structural steel.	7,456,000	5.6	417,536
Bracing, struct. steel..	4,350,000	5.6	243,600
Pins, nickel steel	944,800	10.0	94,480
Total			1,929,267
Cantilever Arm.			
Truss, structural steel.	13,116,200	5.6	734,507
nickel steel	15,931,499	8.9	1,274,512
Floor, structural steel.	6,204,000	5.6	347,424
Bracing, struct. steel..	3,456,400	5.6	193,558
Pins, nickel steel	1,042,000	10.0	104,200
Total	.....	.....	5,308,402
Anchor Arm.			
Truss, structural steel.	11,136,200	5.6	623,627
nickel steel	18,845,000	8.0	1,507,600
Floor, structural steel.	8,272,000	5.6	463,232
Bracing, struct. steel..	3,704,400	5.6	207,446
Pins, nickel steel	894,000	10.0	89,400
Total	.....	.....	5,782,610
Tower.			
Structural steel.....	37,638,000	5.6	2,107,728
Steel castings	2,520,000	9.0	226,800
Total	.....	.....	4,669,056

CANTILEVER BRIDGE NO. 3.

($L = 2000$.)

ESTIMATE OF COST—(*Continued*).

Material.	Quantity. lbs.	Price. cts.	Amount. $
Anchorage.			
Eye-bars and riveted work	3,374,300	5.0	168,715
Total			337,430
Total steel work.			$18,026,765
Main Piers.			
Pier masonry (cu. yds.)	17,860	18	321,480
Concrete filling in cribwork	45,530	7½	341,475
Concrete filling in caisson	6,070	12	72,840
Timber in cribwork (B.M.)	820M	45	36,900
Steel in caisson and shafts	2,701,000	4.5	121,545
Earth excavation (cu. yds.)	45,570	$4	182,280
Total			2,153,040
Anchorages.			
Masonry (cu. yds.)	45,950	$18	827,100
Concrete in foundation	12,075	12	144,900
Total			1,944,000
Total substructure			$4,097,040
Total cost of bridge			$22,123,805

CHAPTER VII

CONCLUSIONS FOR CANTILEVERS

ART. 25

EMPIRIC FORMULÆ FOR WEIGHTS OF CANTILEVER SPANS

LET T_1, T_2, T_3 = truss-weights per linear foot, in suspended span, cantilever arm and anchor arm, respectively; F = weight of floor, per linear foot; LL = live load per linear foot.

We may then construct semi-empiric formulæ for the truss-weights from the results of the preceding designs as follows:

Suspended Span. The weight per linear foot of a simple truss, if the relative dimensions are kept fairly constant, is proportional to the span and to the total load carried. Introducing a factor of proportionality, a, we have

$$T_1 = a(T_1 + F + LL) \cdot l. \quad . \quad . \quad (1)$$

Applying this equation to the preceding designs, we find

$$a = .00056, \quad .00053, \quad .00053;$$

or, as a mean value,

$$a = .00054.$$

Cantilever Arm. The weight of the cantilever arm may be considered as composed of two parts: one due to the reaction of the suspended span applied as an end-concentration, and the other due to the weight and loading of the arm itself. This relation may be expressed symbolically as

$$T_2 = b_1(T_1 + F + LL) \cdot l + b_2 \cdot (T_2 + F + LL)m. \quad . \quad (2)$$

The results of the preceding designs, substituted in the formula, yield

$$b_1 = .00087 \quad .00085 \quad .00087$$
$$b_2 = .00023 \quad .00023 \quad .00023$$

or as mean values,

$$b_1 = .00086$$

and

$$b_2 = .00023$$

A simpler formula, involving but one undetermined coefficient, may be constructed from the observed fact that the weight per linear foot of the cantilever arm varies with the total loading and with the square root of the channel span, or

$$T_2 = b(T_2 + F + LL) \cdot \sqrt{L}. \quad . \quad . \quad (3)$$

The constant of proportionality, b, is determined from the preceding designs as

$$b = .0133, \quad .0134, \quad .0134.$$

The close agreement between these values of b indicates the reliability of the above formula even under varying values of the suspended span ratio. Taking the mean value, we have

$$b = .0134.$$

Anchor Arm. A rational formula for the anchor arm would involve too many factors for practical usefulness. A simple

empirical formula, however, is suggested by the observation that the weight per linear foot of the anchor arm varies with its total load and with the square root of its length. We may therefore write

$$T_3 = c(T_3 + F + LL) \cdot \sqrt{n}. \quad . \quad .(4)$$

Substituting the results of the preceding designs, eq. (4) yields

$$c = .0190, \quad .0189, \quad .0191,$$

or, as a mean value,

$$c = .0190.$$

As the total amount of steel is not materially affected by limited changes in the form or proportions of the trusses, and as variations in the spans and in the loading are fully provided for in the above formulæ, they may properly be applied to any other cantilever bridges designed for any live-load, provided the same working stresses are used. Whenever improved material or construction justifies higher unit stresses,

the empirical constants in the above formulæ should be reduced in the inverse proportion.

Art. 26

Theoretical Limiting Spans for Cantilevers

Maximum Channel Span. A solution of the empiric formula for the weight of the cantilever arm, as established in the preceding article, yields

$$T_2 = \frac{b(F+LL)\cdot\sqrt{L}}{1-b\cdot\sqrt{L}}. \quad . \quad (1)$$

This expression will become infinite when the denominator reduces to zero, i.e., when

$$\sqrt{L} = \frac{1}{b} = \frac{1}{.0134},$$

or

$$L = 5600.$$

Hence *the maximum possible span for a cantilever bridge is* 5600 *ft.* This value may be approached, but never exceeded

by increasing the ratio of dead-load to live-load indefinitely.

Note that the corresponding maximum span for the suspension bridge is 14,700 ft., or about 2.6 times as great as the above.

Maximum Suspended Span. A solution of the empiric formula for the weight of the suspended span, as established in the preceding article, yields

$$T_1 = \frac{a(F+LL)}{1-al} \cdot l. \quad . \quad . \quad (2)$$

This expression will become infinite when the denominator reduces to zero, i.e., when

$$l = \frac{1}{a} = \frac{1}{.00054},$$

or

$$l_{max} = 1850.$$

Hence the maximum possible length for the suspended span is 1850 *ft.* This value has a theoretical significance similar to that of L in the preceding paragraph.

Maximum Length of Anchor Arm. A solution of the empiric formula of the preceding article for the weight of the anchor arm yields

$$T_3 = \frac{c(F+LL)\sqrt{n}}{1-c\sqrt{n}}. \quad . \quad . \quad (3)$$

This expression becomes infinite for

$$\sqrt{n} = \frac{1}{c} = \frac{1}{.0190}.$$

Hence

$$n_{\max} = 2760.$$

The limiting length of anchor arm is therefore 2760 ft.

Combining the above values of limiting spans, we find that the cantilever bridge of maximum length will have the following proportions:

$$l = 1850, \quad m = 1875, \quad n = 2760$$

$$L = l + 2m = 5600, \; S = l + 2m + 2n = 11{,}120.$$

It will be observed that $l_{\max} = 0.35 L_{\max}$. This is less than the economic ratio for ordinary spans and is a justification of the writer's plan of reducing the relative

length of the suspended truss as the total length of span (L) is increased. It is also seen that $n_{max}=0.49L_{max}$, a value approximating the economic ratio.

Art. 27

Theoretical Limiting Span for a Simple Truss

From the weights obtained for the three suspended spans in the foregoing designs ($l=500$, 650, 800 ft.), the limiting spans for ordinary truss bridges may be deduced. A correction must first be applied, however, for the increased stresses produced in some of the members by the cantilever method of erection. A study of the above designs indicates an extra truss-weight, attributable to these erection stresses, of

$$6.3\%,\quad 6.1\%,\quad 6.0\%,$$

respectively, or, as an average, 6.1 per cent of the total weight. Consequently, for a given span, a simple truss would weigh about 6.1% less than the sus-

pended truss of a cantilever bridge. The empiric formula for the latter may then be adapted to the ordinary truss by simply reducing the weight-factor by the above percentage, giving

$$T = .00051(T + F + LL)l.$$

This yields $T = \infty$ for

$$l_{max} = \frac{1}{.00051} = 1960 \text{ ft.}$$

It thus appears that the limiting length for a simple truss is about three times the longest span yet attempted.

Art. 28

Maximum Practicable Span for Cantilevers

The theoretical maximum span established in the preceding article is the limiting span which may be approached by increasing the sections of the truss-members indefinitely. It is evident, however, that long before that span is reached, the required cross-sections will become too great for actual construction.

The longest span at which the cross-sections of the members will not exceed an assigned maximum value, determined by the limitations of design, fabrication, transportation or erection, will be termed the *maximum practicable span.*

In the three cantilever designs executed above, viz.

$$L = 1000, \quad 1500, \quad 2000 \text{ ft.},$$

the maximum required section is found in the lower chord of the anchor arm and amounts to

$$A_{max} = 574, \quad 1302, \quad 2338 \text{ sq.in.}$$

It will be observed that these cross-sections are almost exactly proportional to the *squares* of the respective channel spans. Considering also that the section must vary with the live-load for which the structure is designed, we may write the proportion

$$A_{max} = k(LL)L^2.$$

Substituting the values of A_{max}, LL and L from the above designs, and

solving for the unknown coefficient, we find

$$k = .031, \quad .032, \quad .032,$$

so that the above formula becomes

$$A_{max} = .032(LL)L^2, \quad . \quad . \quad . \quad (1)$$

where L is supposed given in 1000 ft. units.

Hence the maximum practicable span for a cantilever is given by

$$L_{max} = \sqrt{31.2\frac{A_{max}}{LL}}, \quad . \quad . \quad (2)$$

where A_{max} is the greatest practicable cross-section (in sq. in.) and LL is the assumed live-load (in lbs. p.l.f.).

Value of A_{max}. The largest cross-sections ever fabricated are the following:

Queensboro	1120	sq.in.
Quebec	843	"
Beaver	639	"
Kansas City (Truss)	485	"
St. Louis (Truss)	441	"
Monongahela	348	"
Thebes	317	"
Memphis	228	"

All of the above chord-sections are of the multiple rib type. The number of ribs composing the section never exceeds four, that being the number used in the Quebec, Queensboro, Thebes and Memphis Bridges. Any larger number of ribs would make the action of the whole number as a unit extremely problematic; and, in the light of recent experience, no conservative engineer would propose such a section.

The greatest depth of rib is 5′6½″ occurring in the Beaver Bridge, followed by a depth of 4′6½″ in the Quebec and St. Louis Bridges. In Lindenthal's design for the Hell Gate Arch (977′6″ span), a 10 ft. depth of rib is proposed. This is so far in advance of any previous design, and involves so difficult a problem of adequate stiffening and bracing, besides the difficulties of fabrication, that it may well be taken as the extreme limit for practicable depth of rib.

The greatest thickness of rib is 5¾″ in the Queensboro Bridge, followed by

$4\frac{7}{16}''$, $4\frac{1}{4}''$, and $3\frac{1}{2}''$ in the Monongahela, Hell Gate and Quebec Bridges, respectively. On account of the extreme difficulty of riveting through so great a thickness, it is hardly probable that any greater thickness than the above, or say 6″ at the most, will ever be used.

Combining the above limitations, we find that the greatest practicable chord-section would consist of 4 ribs, 10 ft. deep and 6 in. thick. Adding the largest angles obtainable for the flanges, the total area will be, in round numbers,

$$A_{max} = 3000 \text{ sq.in.}$$

With the necessary diaphragms and lacing, such a member would weigh over 7 tons per linear foot.

This value of A_{max} is nearly three times the cross-section of the largest member ever fabricated (1120 sq.in. in the Queensboro Bridge) and over twice the largest section to be used in the Hell Gate Arch (1437 sq. in.). A member of 2300 sq.in. occurs in the plans for the Hudson River Bridge of 2160 ft.

span designed by the Union Bridge Company in 1893, and a slightly larger cross-section (2338 sq.in.) is found necessary in the writer's design of a 2000 ft. cantilever bridge. But the above sectional area of 3000 sq.in. is far in excess of any chord section ever proposed.

Value of L_{max}. Substituting $A_{max}=3000$ in eq. (2) above, we find the following values of the maximum span for different assumed live loads:

For

$LL=10{,}000$ lbs. p.l.f.	$L_{max}=3060$ ft.
$LL=15{,}000$ lbs. p.l.f.	$L_{max}=2500$ ft.
$LL=20{,}000$ lbs. p.l.f.	$L_{max}=2160$ ft.

As it could hardly be considered practical to design a cantilever bridge of the magnitude under consideration for a smaller live load than about 10,000 lbs. per linear foot, we conclude that *the maximum practical span for the cantilever type ranges from about* 2000 *to* 3000 *ft., depending upon the assumed live load.*

Compare these with the corresponding

values established above for the suspension bridge, viz.,

$$l_{max} = 3500 \text{ to } 4900 \text{ ft.},$$

indicating the greater suitability of the suspension type to extreme long-span construction.

Art. 29

Empiric Formula for Cost of Cantilever Bridges

As in the case of suspension bridges, the expression for the cost of any span (L) will be assumed of the general form:

$$C = aL + bL^2 + cL^3. \quad . \quad . \quad . \quad (1)$$

Since we have but three values for C determined by actual design, the formula is limited to an equal number of terms.

From the results of the preceding estimates, we have

$C = \$4{,}905{,}000$	for $L = 1000$ ft.
$C = 10{,}570{,}000$	$L = 1500$ ft.
$C = 22{,}125{,}000$	$L = 2000$ ft.

Substituting these values in eq. (1), the unknown coefficients are determined, giving

$$C = 6350L - 5.25\,L^2 + .0038L^3, \quad (2)$$

as the general cost-formula for cantilever bridges. This gives the cost of steelwork and substructure for an assumed live load of 18000 lbs. per linear foot. For any other loading, the cost may be taken as varying in proportion.

Art. 30

Economic Span for Cantilevers

As established in the preceding article, the cost of a cantilever bridge, including steelwork and substructure, is given by the expression

$$C = 6350\,L - 5.25L^2 + .0038L^3 \quad (1)$$

Assuming the same cost of approaches, volume of traffic, life of steelwork, expense of maintenance and rate of interest as for the suspension type, the limiting economic span for the canti-

lever will be defined by the same total cost as in the case of the suspension bridge, viz.,

$$C = \$54{,}000{,}000.$$

As shown in Art. 19, this is the greatest expenditure for a long-span bridge, exclusive of approaches and accessories, that is warranted by the maximum traffic returns to be reasonably expected. Substituting this value in eq. (1) and solving we find

$$L = 2700 \text{ ft.}$$

as the maximum economic span for cantilevers.

Hence, the greatest span for which the cantilever type may be profitably employed is 2700 ft. or less, depending upon the probable maximum traffic returns.

CHAPTER VIII

FINAL COMPARISONS AND CONCLUSIONS

ART. 31

COSTS OF SUSPENSION BRIDGES AND CANTILEVERS

In the accompanying table are given the costs for various spans of suspension bridges and cantilevers as determined by the respective empiric formulæ. The following relations may be gleaned from the table:

1. The cantilever is cheaper than the suspension type at a span of 1500 ft. or less. The suspension is cheaper than the cantilever type at a span of 1750 ft. or more.

2. The cost per foot of length for suspension bridges is a minimum for a span of about 1000 ft., indicating a rapid decrease of economy in the use of the type for shorter spans.

COSTS OF SUSPENSION BRIDGES AND CANTILEVERS.

for $LL = 18{,}000$ lbs. p.l.f.

Span $= l$	Suspension Bridges. Total Cost $= C$	Cost per Foot. $= C \div 2l$	$K = C \div l^2$	Cantilever Bridges. Total Cost $= C$	Cost per Foot. $= C \div 1.8l$	$K = C \div l^2$
	$	$		$	$	
250	2,023,000	4,040	32.3	1,343,000	2,980	21.6
500	3,758,000	3,760	15.0	2,288,000	2,540	9.2
750	5,395,000	3,595	9.6	3,410,000	2,525	6.1
1000	7,130,000	3,565	7.1	4,905,000*	2,725	4.9
1250	9,140,000	3,655	5.8	7,035,000	3,130	4.5
1500	11,645,000*	3,880	5.2	10,570,000*	3,910	4.7
1750	14,130,000	4,040	4.5	15,370,000	4,880	5.0
2000	18,720,000	4,680	4.7	22,125,000*	6,150	5.5
2250	23,760,000*	5,280	4.7	30,990,000	7,640	6.1
2500	29,950,000	5,990	4.8	42,570,000	9,450	6.8
2750	37,390,000	6,790	4.9	56,910,000	11,500	7.5
3000	46,785,000*	7,780	5.2	74,650,000	13,850	8.3
3250	57,710,000	8,870	5.5	95,740,000	16,360	9.1
3500	71,050,000	10,160	5.8	121,120,000	19,240	10.0
3750	85,780,000	11,450	6.1	150,710,000	22,300	10.7
4000	103,200,000	12,900	6.4	184,900,000	25,680	11.5

$C = 8900l - 3.77l^2 + 0.002l^3$ or $C = K.l^2$ $\quad$ $C = 6350l - 5.25l^2 + .0038l^3$ or $C = K.l^2$

*** Values obtained by actual design. Other values interpolated or extrapolated by aid of empiric formulæ given below.**

3. The cost per foot of length for cantilevers is a minimum for a span of about 600 ft., indicating a rapid decrease of economy in the use of the type for shorter spans.

4. Between the limits of 1500 and 3000 ft. for suspension bridges, and of 1000 and 1750 ft. for cantilevers, the value of $K(=C \div l^2)$ remains fairly constant. Hence, for all normal spans, the cost of either type may be estimated as varying approximately with the square of the span. Within the above ranges, as an average value,

$C = 4.7l^2$ for suspension bridges

and

$C = 4.8\ L^2$ for cantilevers.

5. The value of K for the suspension bridges is a minimum for a span of about 1800 ft., indicating that that is the span for which the suspension type is economically best adapted.

6. The value of K for the cantilevers is a minimum for a span of about 1250 ft., indicating that that is the span for which the cantilever type is economically best adapted.

7. The limiting economic span, i.e. the greatest span for profitable erection, defined by $C = \$54{,}000{,}000$, is shown

by the table to be about 3200 ft. for the suspension type and about 2700 ft. for the cantilever type.

The tabulated costs, where they lie outside of the spans actually designed, are subject to the inherent errors of any method of extrapolation; but the above results are sufficiently accurate for the purposes of this investigation.

Art. 32

Span of Equal Cost

Plotting the costs for the three suspension spans designed above, a smooth curve passing through the three points and the origin constitutes a cost-graph for suspension bridges. In the same manner, the graph representing the costs for different cantilever spans is constructed. The point where the two curves intersect marks the span of equal cost for the two types of construction, and is found at

$$l = 1670 \text{ ft.}$$

Below this span, the cantilever bridge is cheaper; above this span, the suspension bridge exhibits the greater economy. As this value lies within the range of spans actually designed for each bridge, i.e., between the known points on each graph, the error of the above method, involved in extending the results of individual designs to other spans, is negligible.

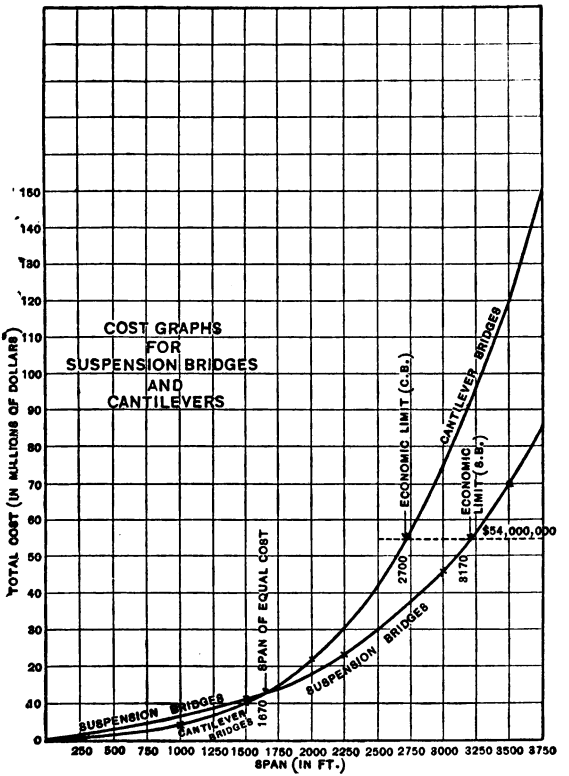
The analytical equivalent of the above process consists in comparing the cost-formulæ for the two types of bridges. The cost of a suspension bridge, designed for a live-load of 18,000 lbs. per linear foot, as established in Art. 19, is given by

$$C = 8900l - 3.77l^2 + .0020l^3, \qquad (1)$$

and that of a cantilever bridge for the same loading, as established in Art. 39, by

$$C = 6350l - 5.25l^2 + .0038l^3, \quad . \qquad (2)$$

where l denotes the total channel span. Equating expressions (1) and (2), and


COST GRAPHS
FOR
SUSPENSION BRIDGES
AND
CANTILEVERS
TOTAL COST (IN MILLIONS OF DOLLARS)
SPAN (IN FT.)
SUSPENSION BRIDGES
CANTILEVER BRIDGES
SPAN OF EQUAL COST
1670
ECONOMIC LIMIT (C.B.)
2700
CANTILEVER BRIDGES
ECONOMIC LIMIT (S.B.)
3170
$54,000,000
SUSPENSION BRIDGES

solving, we find, for the span of equal cost,

$$l = 1670 \text{ ft.}$$

exactly as before.

The above comparison is somewhat unfair to the suspension type, as the side-spans in the above designs were $0.5l$ in the suspension bridges and only $0.4l$ in the cantilevers. Assuming, therefore, that steel viaduct approaches are added to the cantilever bridges to make up the difference in total length, and estimating the cost of such viaducts at $1000 per linear foot,[1] the expression (2) becomes modified to

$$C = 6550l - 5.25l^2 + .0038l^3. \quad . \quad (3)$$

Equating this to expression (1) and solving for the span of equal cost, we find

$$l = 1626 \text{ ft.},$$

somewhat less than the value established above. If the comparison of costs had

[1] This is the price assumed in a similar case by the U. S. Board of Engineers in 1894.

been made between bridges of equal total length instead of between bridges of equal channel span, the result would have been still more favorable to the suspension bridge, reducing the critical span to about

$$l = 1500 \text{ ft.}$$

Neglecting these differences in favor of the suspension type, we conclude that $l = 1670$ ft. is the extreme upper limit of spans at which the cantilever can compete with the suspension bridge, when economy is the sole criterion.

Art. 33

Summary

The results of the preceding investigations may be summarized as follows:

1. *Maximum Span for a Cable.* The greatest span theoretically possible for a steel cable of any cross-section is 65,520 ft. based on the ultimate resistance; 21,840 ft. based on a safe working-stress of 60,000 lbs. per sq.in.; 15,160 ft.

if the rise is restricted to the economic ratio of one-eighth the span; and 14,700 ft. if the weight of cable-wrapping and fastenings is taken into consideration.

2. *Maximum Span for Suspension Bridges.* The last value, 14,700 ft., is also the maximum span theoretically possible for a stiffened suspension bridge. The greatest practicable span, defined by a maximum section of 16 cables of 24 inches diameter with a minimum live-load of 10,000 lbs. per linear foot, is 4300 ft.

3. *Economic Span for Suspension Bridges.* The greatest suspension span for which the necessary outlay would be warranted by the probable traffic returns is 3170 ft.

4. *Maximum Span for Cantilever Bridges.* The greatest span theoretically possible for a cantilever bridge is 5600 ft. In this maximum span, the suspended span will be 1850 ft., the cantilever arms 1875 ft., and the anchor arms 2760 ft. The practical limit for cantilevers, defined by a maximum chord-

section of 3000 sq.in., with a minimum live-load of 10,000 lbs. per linear foot, is 3060 ft.

5. *Maximum Span for Truss.* The greatest span theoretically possible for a simple truss is 1960 ft.

6. *Economic Span for Cantilever Bridges.* The maximum economic span for cantilevers, defined by the condition of zero net profit on the investment, is 2700 ft.

7. *Span of Equal Cost.* The critical span at which the suspension bridge becomes economically superior to the cantilever bridge is 1670 ft.

Summary:	S. B.	C. B.
Theoretical max. span..	14,700	5,600
Practical max. span....	4,900	3,060
Max. economic span....	3,170	2,700
Span of equal cost	1,670	

Art. 34

Conclusions

In the foregoing designs, special care was taken to proportion the depths of truss in both types for equal and ample

rigidity, so that the single inherent advantage claimed for cantilevers, viz. greater stiffness for railway traffic, is eliminated from consideration. Consequently no advantage remains to the cantilever type above the limiting span of 1670 ft. The suspension bridge, on the other hand, is universally admitted to possess greater aesthetic qualifications; and, for a long-span city structure, this is a factor of decisive importance. For this reason alone the above value for the span of equal cost should be considerably reduced in favor of the suspension type, to obtain the "span of equal merit."

The preceding investigations have been restricted to designs for heavy railway traffic. For highway bridges, a relatively lighter stiffening truss may be used in the suspension type, thereby causing a considerable reduction in the span of equal cost.

Confining our attention to economic considerations, our final conclusions may be stated as follows:

1. *The range of economic usefulness for cantilevers extends from the upper limit for the truss or arch to a span of* 1670 *ft. Beyond this value, the cantilever would be more costly than the suspension type, although yielding a probable profit on the investment up to a span of* 2700 *ft.*

2. *The range of economic usefulness for suspension bridges begins at* 1670 *ft. (or less in the case of highway bridges) and extends to the upper economic limit of* 3170 *ft. Above this limit, the construction of suspension bridges would be practically feasible, but not as a profitable investment, up to an extreme limit of* 4900 *ft.*

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